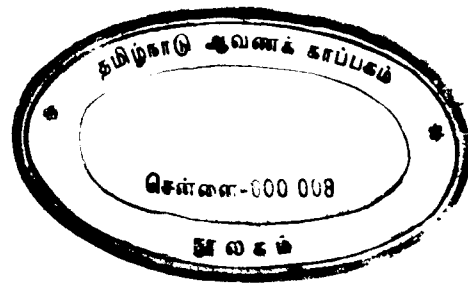


L.C.E. 34

A MANUAL OF
CIVIL ENGINEERING PRACTICE.



C.E. 34



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PREFACE.

THIS book is designed to meet the need of young engineers who are preparing to occupy positions of responsibility such as "Municipal and County Engineers," "Borough Surveyors," etc. It deals with the general practice of municipal engineering work and the theoretical principles involved therein, and includes such subjects as Design of Structures in brick, stone, and reinforced concrete; Chimneys, Steel Construction, and Mechanical Engineering; Roads, Electric Tramways, Bridges, River work, and Land Drainage; Hydraulics and Pumping Machinery; Waterworks and Sources of Supply; Sewage Disposal, Filtration, and Purification; Sanitation, House Drainage, and Disinfection; Refuse disposal; also Specification.* The information has been gathered for the most part from the author's own experience in practice.

Succeeding generations of engineers have found Professor Rankine's book upon Civil Engineering of the utmost value, and, as revised from time to time, this book has held and continues to hold an unchallenged place. But in relation to modern municipal engineering practice the author of this present text-book ventures to hope his work may prove of value, and in many ways supplemental to that of Professor Rankine.

Young men qualifying for the engineering profession, and who have had the advantage of a University education, often lack just that training which used to be secured under the old system of apprenticeship, and to such as these the record of experience given in these pages should be of service. The attempt is here made to present under one cover a compendium of Municipal Engineering without the work becoming a voluminous encyclopædia, and to accomplish this the author has relied upon a large number of specially prepared illustrations which do away with the necessity of extended text, as every engineer should be able to read these drawings with the same ease as he can read the text.

So varied are the duties devolving upon the Municipal Engineer of to-day that a very wide range of knowledge is required to fulfil them; but the mind cannot retain every detail, and reference to such a work as this becomes

* The subjects of Gas and Electrical Engineering are only touched upon as they relate to the duties of the surveyor.

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SPECIALLY ARRANGED FOR THE USE OF

MUNICIPAL AND COUNTY ENGINEERS.

L.C.E. 34

BY

F. NOEL TAYLOR, C.E., M.Inst.M.E.

With 1179 Illustrations, including 35 Folding Plates.

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CIVIL ENGINEERING PRACTICE.

CHAPTER I.

ORDNANCE MAPS.

The first and most prominent objects in a Civil Engineer's office are his maps, and a few words on these important accessories will therefore not be out of place as an introduction to what is to follow.

It is absolutely essential to every engineer, surveyor, and estate agent to have good and reliable maps of the particular districts with which he is concerned, as when once these are provided, accurate and up-to-date in every detail, they become an engineer's most valued aid. They save him time and money, and are of the utmost assistance in legal cases.

Maps of estates should be coloured to show the different holdings, each holding being carefully numbered and referred to in a schedule, where the fullest information, which it would be impossible to print on the map itself, is given. For town and country work they should have all watercourses carefully marked; all sewers, drains, water and gas mains, fountains, electric cables and public conveniences very carefully indicated; in fact, all possible information which a surveyor has handy should be neatly entered thereon, various colours, properly used, being of the utmost assistance in accomplishing this.

These maps may be the results of extensive surveys which have taken years to complete, or they may have been plotted from various sources, such as old maps, which were made with remarkable accuracy considering the disadvantages under which surveyors laboured in olden days, but in an up-to-date British engineer's office they are usually what are termed *Ordnance maps*. These Ordnance maps are the result of the Ordnance Survey of Great Britain and Ireland, first undertaken by the Royal Engineers* in 1747, intended for military purposes, but now carried on by the Board of Agriculture and Fisheries. These maps are marvels of accuracy and cheapness. They are on sale in most towns, but the head offices are at Southampton and Dublin, and the price varies from $\frac{1}{2}$ d. per sheet upwards, according to size and district.

The scales upon which they are printed are—

- 25380, or 1 inch to 1 mile. = 1 inch map
- 10750, or 6 inches to 1 mile. = 6 inch map
- 4300, or 25 inches to 1 mile. = 25 inch map

* See White's *Ordnance Survey of the United Kingdom*, 1886.



FIG. 41.—Example of Twenty-five-inch Map.

The latter are usually known as "25-inch maps," and are especially useful for estate work, and town and country work where the ground is moderately open and there is not too much detail. In addition to the foregoing, there are obtainable 5-foot maps (scale 5 ft. to 1 mile), and in most towns of over 4000 inhabitants the 10-foot map (126.72 inches to 1 mile), commonly called the 10-foot map.)

The sheets usually measure 40" x 27", but they vary.

(Of all the maps published, perhaps the 25-inch and 5-foot maps are used most) therefore it would be well to illustrate by examples to scale these two varieties, fig. 1 being an example of the 25-inch and fig. 2 an example of the 5-foot scale.

When about to prepare maps of a district, it is absolutely necessary to note the dates on the sheets, because they are very often considerably out of date, although revised from time to time. Therefore the first thing to do is to put in all new buildings, roads, and streets as accurately as possible. For this purpose a set of sheets should be procured, and the parts checked by going over the ground bit by bit. It is most important that a surveyor should have experience in this sort of work, as it often happens that the maps of many of the smaller towns are in a deplorable state. After having revised the sheets as far as possible, a second set should be procured and an exact copy made, because by the time the revision is finished the original map will be fairly dirty. When completed the map should be sent away to a firm accustomed to such work, carefully cut and mounted on linen, and placed on a roller to hang up on the wall. We then have a map the value of which is only known to the engineer who has taken the trouble to make one.

As I have already pointed out, these maps should have all the information obtainable marked thereon, such as sewers and water mains, valves, hydrants, manholes, etc.; but as a single map would be likely to be overcrowded with so much detail, especially in populous districts, a separate map showing sewers and water mains would be an advantage. As gasworks and electric power stations are usually in charge of separate managers, it is hardly necessary to refer to these subjects here. Of course there are special terms and abbreviations to be used in connection with such special maps, but we shall refer to these and give examples when we come to deal with main drainage and water distribution.

(In very large districts with practically open country the 6-inch map is largely used under similar conditions, and especially for preliminary surveys and for parliamentary plans.) They have the contours marked thereon, that is, lines of equal altitude, the uses of which will be explained further on. It will also be observed that on the maps certain abbreviations occur, such as H for hydrant. The meaning of such abbreviations is usually fairly obvious, but a list of the most frequently used may not be out of place. They are—

H = hydrant.
LP = lamp-post.
Chy = chimney.

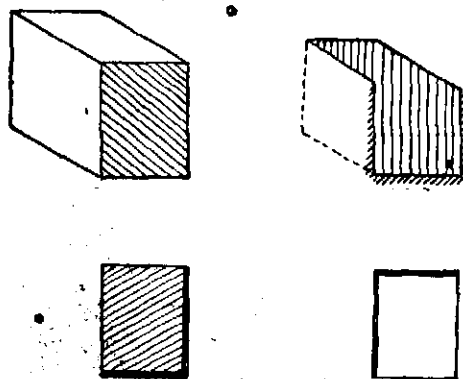
TP = telegraph pole.
MP = mooring post (canals and docks).

△ means that there was a theodolite station here at 1296 ft. above Ordnance datum.

And the most important of all—BM 92.7. This is the "bench mark," which indicates one of the fixed points from which data employed in the plotting of the Ordnance Survey maps are derived.

and tedious preliminary survey, the above principle is very expeditious. It applies not only to roads, but to railways, to the laying of pipe lines, drains, and sewers. The vertical interval of contours on maps will of course vary with the scale of the map. In the old 5-foot maps, they were 5 ft. apart, but on the 6-inch they are usually 50 ft. on fairly stiff ground, but less in very flat districts.

In concluding these few remarks on maps I need scarcely add that accuracy is the main thing, and neatness and conformity in printing and colouring second only to it. A question which frequently confronts the young engineer is, why have the buildings and other parts of the map thick and thin lines? The explanation is this. The map maker considered



Shade Lines
Representing Projection

Shade Lines
Representing Depression

FIG. 8.

the light as coming from the top left-hand corner of the map. He therefore assumed that the right-hand side and bottom side of any building were more or less dark or in the shade. He therefore put thick lines here, showing a projection. If, however, the reverse should occur, that is, lines facing the imaginary ray of light be thick lines, then there is a depression here, such as a well, cellar, or watercourse. A little study of a map will soon show the reader that this is so. They are what are called shade lines, and, when one is accustomed to using them, make the map very clear (fig. 8). One can, if in doubt, distinguish between a road and a watercourse by means of these lines. In the same way, on all good engineering drawings shade lines occur, with the object of defining projections and depressions.

A special index is issued by the Ordnance Survey, for use with the 25-inch Ordnance map. It gives much useful information. The maps show the position of practically every farm in a parish, and the boundaries and names of every parish represented in the index. Every parcel of land is numbered and its area computed, the area being given beneath the number on the map.

A catalogue of maps is issued by the Director of the Ordnance Survey, and the maps themselves may be obtained through the local post office, and many of them are, indeed, kept in stock at the head offices of many of the larger towns.

CHAPTER II.

ON SURVEYING WITH THE CHAIN.

SURVEYING is the term given to the art of measuring up and representing truly to scale on paper the actual configuration of a particular portion of the earth's surface considered as a plane. The art has been practised since very ancient times. Owing, however, to the excellent edition of the Ordnance maps of the United Kingdom, the practice of extensive surveys has now been largely abandoned. Such work is rarely, if ever, now carried out in this country except by Ordnance men, whereas persons who have similar work to do abroad, and particularly in new countries, are usually specialists in their work and do little else. Surveys nowadays are usually of a more restricted nature, conducted generally with the aid of good maps, and for the purpose of waterworks, sewage works, railways, roads, and drainage schemes, and the sale and conveyance of landed property. Surveys abroad

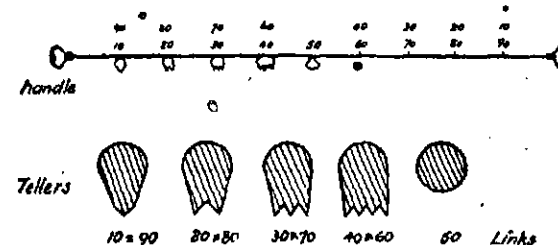


FIG. 9.—Surveying Chain.

in undeveloped and unmapped country are carried out on such entirely different lines to those at home that it would be foreign to the scope of the present work to treat of these here.

Small surveys in the country are often conducted by means of the chain only. This statement, however, only applies to the survey of very open country. When at all complicated, a theodolite has to be used if anything like accuracy is desired and time is a secondary consideration.

As a preliminary to survey proper, a rough sketch should be made in the "field book" showing the nature of the ground. This will serve to show where best to run chain line. Having then carefully reconnoitred the ground we can start chaining. All surveys are made on the principle of "triangulation," that is, of setting out the ground in triangles. These triangles should be as nearly equilateral as possible. Before going any further, however, it

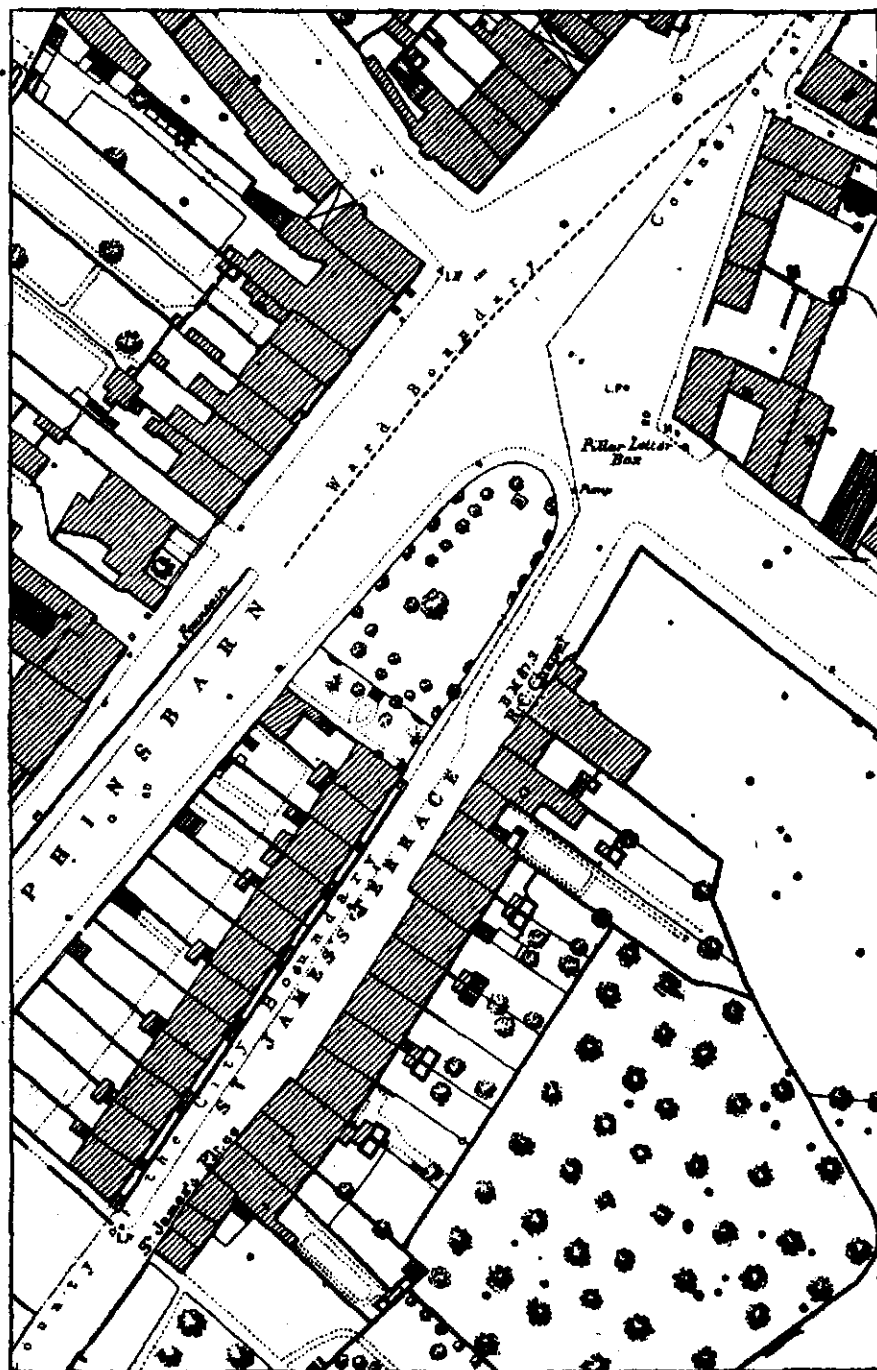


FIG. 2.—Example of Five-foot Map.

There will also appear sometimes on the 5-foot map, as shown in fig. 2, small crosses (+) accompanied by numbers. This + means that the point on the road or footpath was so many feet above the Ordnance datum line (the level of Old Dock sill at Liverpool).

A suggested new road or other line of communication affords a problem frequently confronting the surveyor of the present day. As, however, such suggestions at the first stages are only of a preliminary character, it is not usual to devote much time to level and theodolite observations of the proposed site. A much more expeditious way is to use the 6-inch Ordnance Survey maps, which usually show the "contours" by means of fine dotted lines, as in fig. 3, with figures marked on them as shown. These figures are

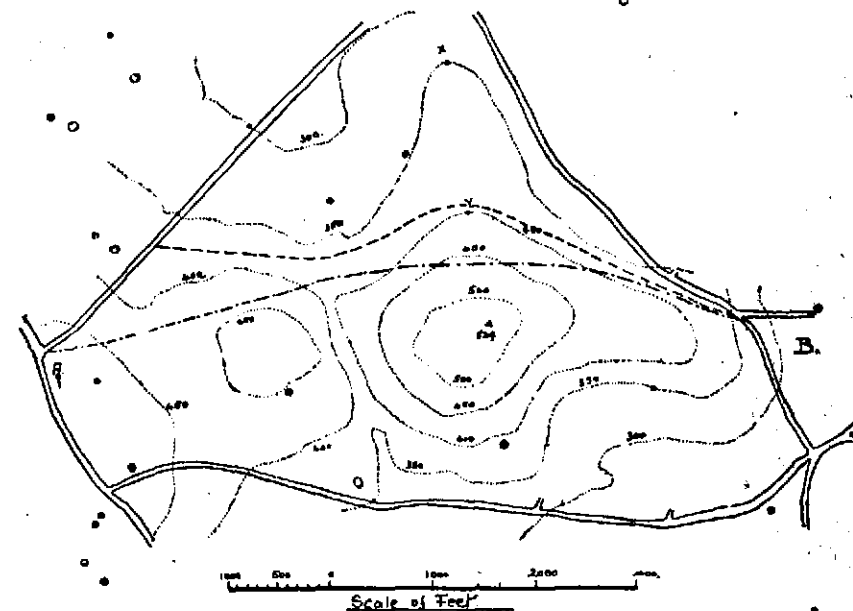


FIG. 3.

the heights above Ordnance datum. These contour lines connect all points which have the same altitudes or depressions, the different lines being numbered to indicate the height or depth referred to datum. Thus the 65 contours would be 65 feet vertically above Ordnance datum, and all points on that line would be at the same height and therefore on the same level.

The datum line is for England on the Old Dock sill at Liverpool, and in Ireland a point 21 feet below a mark on Poolbeg Lighthouse in Dublin Bay. Those points, however, do not concern us. We very often see on a gatepost or bridge a mark so ∇ . By reference to the Ordnance map of the district we shall by finding the point in question see, say, BM 65.4. BM stands for bench mark, the term applied to these marks, while the figures mean that the horizontal bar is 65.4 ft. above Ordnance datum. These marks are of inestimable value to surveyors, and are the usual starting-point for a series of levels. Levelling, however, is not what we want to explain

here; we want to point out how to avoid it by judicious use of contours. Suppose that the construction of a road be proposed between the points A and B in fig. 3, the approximated distances between them being 8000 ft. The three main factors to be taken into consideration are—

1. Limiting gradient.
2. Cost of overcoming natural obstacles.
3. Reduction of the distances to a minimum.

A 6-inch Ordnance map should be procured and studied until the contours are thoroughly understood. The keen observer will no doubt often notice

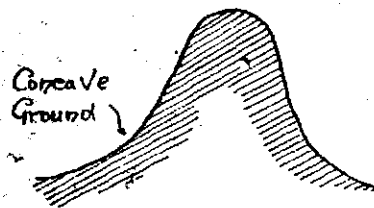


FIG. 4.



FIG. 5.

a small enclosed circle, as our example the 450 contour shows, and that perhaps the lines are nearer together at the top of the hill (viz. inside the circular line) than the bottom. In such a case the ground is, as in fig. 4, concave; if the reverse is the case, and the lines are closer at the bottom than the top, the ground would be as in fig. 5. This rule has to be borne in mind if we wish to make a profile as in fig. 6. Now the profile in question is made to an exaggerated scale, so as to show with sufficient clearness the irregularities of the ground. The horizontal scale is the same as in fig. 3.

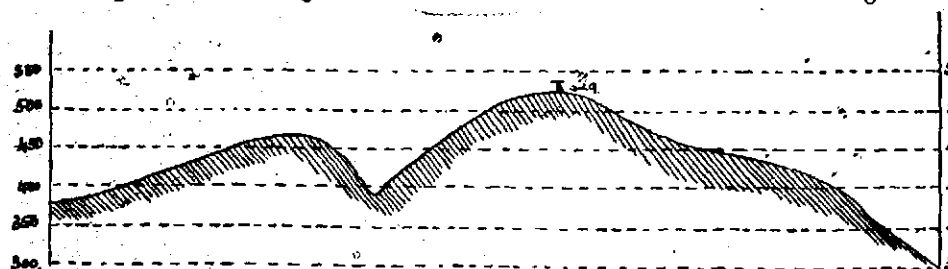


FIG. 6.

The profile is made along the straight line A and B, fig. 3. The detail is then filled in as nearly as possible by freehand. Bearing in mind what was said before in reference to the distance between the contours denoting convex or concave ground, at 529 ft. above O.D. we have a fixed point, because in fig. 3 it will be noticed that there is a mark Δ . This means that there was a theodolite station there during the progress of the Ordnance Survey. The figures beside it denote the actual height.

We now have to see how to lay out an economical road between A and B. On any road a gradient of 1 in 20 is quite steep enough, and steeper gradients should be avoided. These gradients may be expressed as 1 in 30, 1 in 40, or a 1-degree, 2-degree slope, etc. Anyone looking up a table of sines,

agents, etc., will find that the natural cotangent of 1° is 57.3; we therefore infer that an angle of 1° gives a slope of 1 in 57.3. For practical purposes we call this 1-60. Therefore 1-30 would be a 2° slope, and 1-20 a 3° one, and so on. Thus, given our contour

map as in fig. 3, we may take any two points such as x and y. We know that the vertical interval is 50 ft., and by scaling off we find the distance xy = 1450. Now $\frac{50}{1450} = \frac{1}{29}$, practically a 2° slope. We can therefore infer that the ground rises moderately. Such a repetition of calculations would, however, be a tedious affair, to avoid which we can make what is termed a scale of slopes.

As the vertical interval is 50' and 1° slope = a gradient of 1 in 57.3 we are making these for permanent use, we had better say 57.3 instead of 60, and so secure accuracy once and for all, we shall, to rise 50' on a slope, want $50 \times 57.3 = 2865$ ft. horizontal; on a slope of 2° , 1432.5 , etc.; and by similar reasoning we obtain our scale of slopes. Fig. 7 shows the same, constructed to scale for the 6-inch maps. By selecting two points on any two contours and finding between them the nearest distance on the scale of slopes we can read off the gradient at a glance. Now, in reference to fig. 3, and in order to choose the best route for a road between A and B, the ruling gradient being taken at 1 in 30, by use of the scale of slopes we find that the dot-and-dash line satisfies all requirements; or if we have not enough data to satisfy us making such a scale, take as before a gradient of $30 = a 2^\circ$ slope. Now 28.6 is the natural cotangent of 2° , $\therefore 50 \times 28.6 = 1430$ ft. Take 1430

horizontal on the dividers and step from contour to contour. By these means we obtain a line similar to the dot-and-dash line in question, which appears to answer our purpose well. But by again inspecting the map that the dash line runs practically along the line of the contour (400). Therefore this line is practically level, but was avoided owing to the very stiff gradient on the road running northward from A. By the examples given, it will be seen that for those who have to prepare road schemes, and who do not want the trouble of an expensive

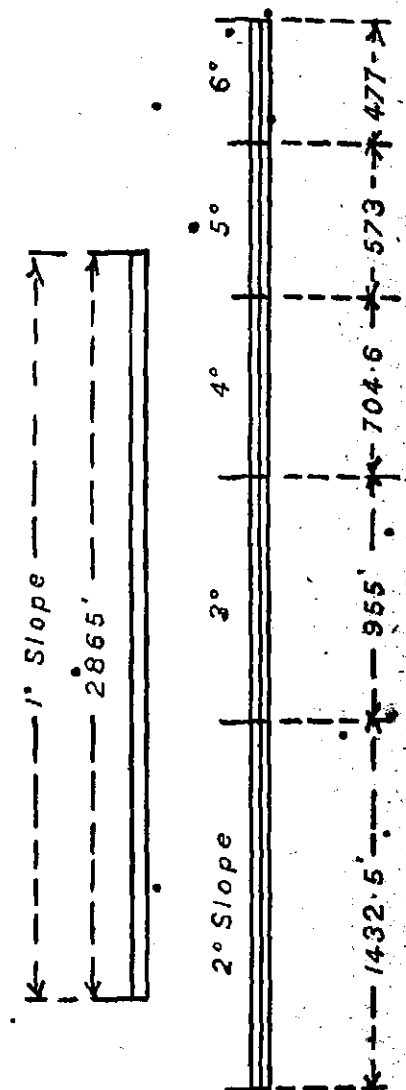


FIG. 7.

is desirable fully to describe the surveyor's implements, such as the chain, arrows, etc.

The 66-foot chain, commonly known as Gunter's chain, was invented by Edmund Gunter in 1620. It is divided into 100 links. Each link measures 7.92 inches. The reason for using a 66-foot chain is because it is an exact decimal part of an acre and a mile, that is, 80 chains = 1 mile, and 10 square chains = 1 acre. This latter must not be confused, however, with 10 chains square, that is, $10 \times 10 = 100$ square chains = 10 acres. The 66-foot chain is chiefly confined to estate work and railways.

For general surveys, town surveying, and county engineers' work a 100-foot chain of 100 links is much more useful and a less likely source of error.

Upon examining a chain we find two brass swivel handles, one at each end, as shown in fig. 9, and brass tablets at equal intervals of 10 links, those at 10 and 90 being similar, and likewise those at 20 and 80, 30 and 70, 40 and 60. At 50 links a circular tablet is inserted. This may puzzle the beginner at first, and until he gets quite conversant with the chain he should be very careful to make sure his reading is correct, and to think twice as to which end he is reading the distance from. It is with the object of being able to read from both ends of the chain that the brass tablets,

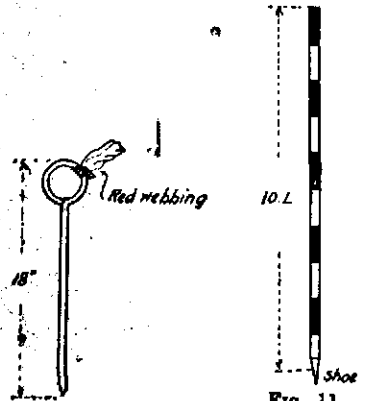


FIG. 10.—Arrow:

FIG. 11.—Ranging Pole.

or "Tellers," as they are called, are arranged so. When purchased, all chains should read within $\frac{1}{4}$ in. of exact measurement, and where constantly used they must be frequently tested either by means of a steel tape kept for the purpose or on known public testing-places. These occur in most large town or public buildings, such as those at the Guild Hall and on the north side of Trafalgar Square in London, and the City Hall in Dublin. Alternatively, a surveyor may make permanent marks once and for all on the plinth of a public building, or on a footpath or other suitable spot.

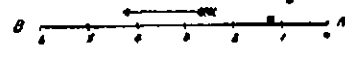
Chains are of two qualities, "strong" and "light." The former are made of 8 BWG and the latter of 12 BWG steel wire.

Arrows are made from the same gauges of wire as the chain (fig. 10). They are made in sets of 10; and 10, and 10 only, must be used on any survey. They are about 18 in. long, and usually have a strip of red webbing tied to them to make them easily visible.

Surveyors' Rods.—These are shown in fig. 11, and have an iron ferrule at one end, and are painted alternate black and white in lengths of a link. They are used to put the chain men in line for taking sights by means of a theodolite or other instruments, and for measuring offsets.

The foregoing appliances, together with a 100-foot tape line, which is best made of Scotch linen, form the necessary equipment for a chain survey. To

about a survey, two men should always be employed—where possible a



chain line

FIG. 12.

leader and a follower. Some try to survey with only one assistant, but it is bad policy, because there is quite enough for one person to do filling in the

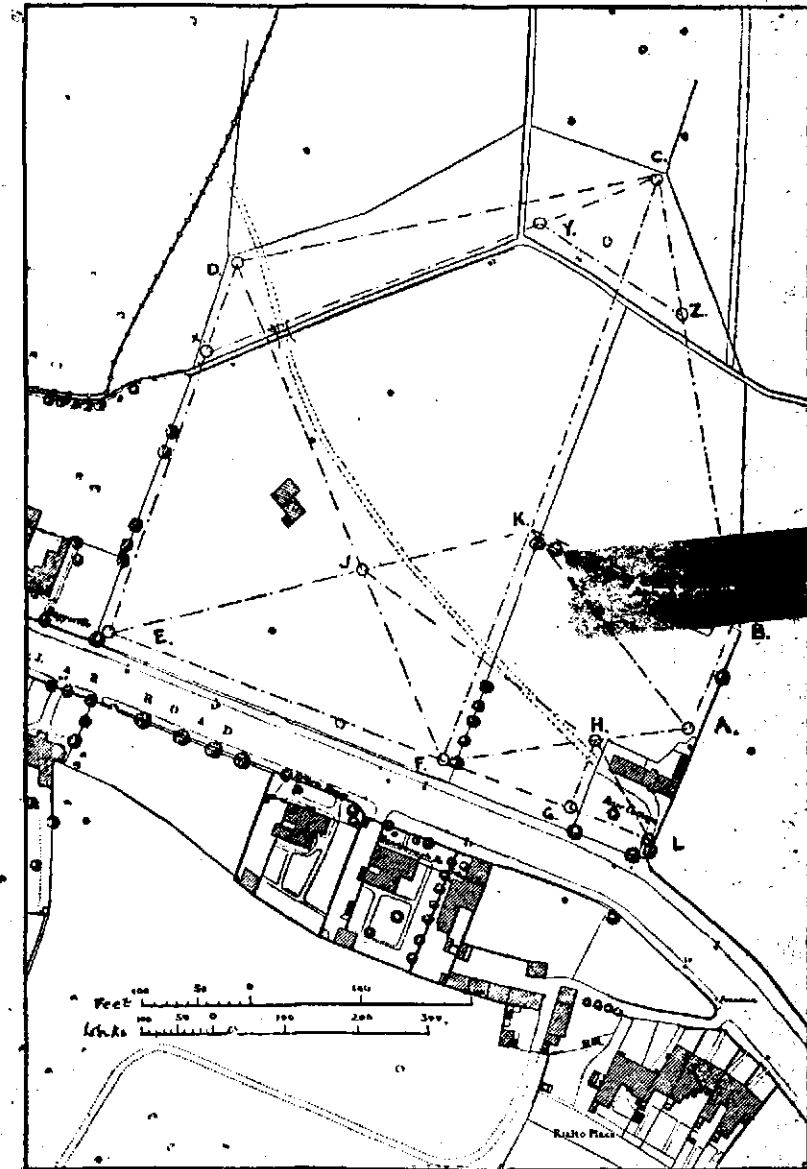


FIG. 13.

field book, and to a great extent the success of a survey depends on the accuracy and neatness of the field book.

We will now suppose that everything is in readiness for making the survey, and that the surveyor has decided upon where to take his main chain links, and marked their position upon his sketch. It now only remains to explain the reason for the 10th arrows mentioned above. Say we are chaining straight (and all chain lines must be quite straight) between two points A and B, fig. 12. The follower holds the chain at A and puts the leader in line to right or left till the chain lies in the direction AB. The chain must be shaken to get out any twist. When quite flat an arrow is put in by the leader, who starts with 10, and another chain length measured off, and so on. In the meantime the follower has picked up the arrows, and when the end of the line is reached the entries in the surveyor's field book should agree with the number of arrows held by the follower. It is most important to attend to this precaution, and also not to put in an arrow at the starting-point, as that throws the whole calculation one out. The odd links must, of course, be read off in the usual way.

Referring to fig. 13, we will assume the usual rough sketch has been made, and it has been decided that the best lines to make chain lines are A-B, B-C, C-D, D-E, EF, FG, GL, F-A, F-C, K-B, H-G.

In order to plot the triangles, and as we are not measuring any angles we must have three sides, therefore it will be necessary to chain the lines E-K, D-F, J-H, H-L, A-K.

While DJ and JH will be necessary in any case to plot the footpath, it would also be advisable to take two subsidiary chain lines XY and YZ, and to plot the watercourse. While measuring each line offsets are taken to the nearest objects, such as fences, walls, buildings, etc. All offsets must be perpendicular to the chain lines and as short as possible. If it should be



FIG. 14.—Setting a Right Angle.

necessary to set off a right angle, this may be done with the chain as in fig. 14, by setting up along the chain 50 links, then 30 links and 40 links, as shown. We then have a right-angled triangle. Some use such objects as optical squares, cross-staffs, etc. In the author's opinion these are only a source of error, any ordinary offset lines being easily set off by the eye or by the method stated.

Coming now to the field book, we shall assume the chain line BC is being measured, and that a 66-foot chain is being used. Always start from the bottom of the page. Take a fresh page for each line, and put on that page all the information handy.

To take an actual example. Suppose we start (page 16) at A and take the short chain line AB.

All the distances along the line appear in the centre column and start from "0" upwards; all lines should also be as long as possible.

The other objects on the ground appear to the right or left of the chain line column in the book, according to the side of the observer they are on. That is why we start at the bottom of the page and work upwards. Always try and make all objects appear as nearly as possible as they are on the ground. Proceeding with our field book, at the station, the only object we have is line AH, going off to the left. Draw it so. At 156 links we cross a fence, and at 170 we come to the end of that particular line, and state that lines BC and BK start here. In the meantime we have taken offsets at right angles on the right side at 0, 156, and 170, and find the distances to the contiguous wall to be 15, 16, and

16 links at the various points. Continuing in the same way, line BC is plotted.

At 381 links we cross a stream, which is obviously 391 - 381, 10 links across.

At 428 we come to the beginning of line ZY. A circle is drawn round the figure and all the chainages which refer to the interposition of other chain lines. At these points it is advisable to drive in pegs when the ranging rods which were placed there to put chain men in line have been drawn.

The offsets are measured in the same way. The process is the same right through the survey, and it is needless to give further examples of the field book, as with all similar work the *sine qua non* is, as usual, practical experience. The principle is simple enough, and those who are inexperienced should practise such work until perfect. No better practice can be obtained than by going over a piece of land similarly to that described, making up the field book and plotting the survey.

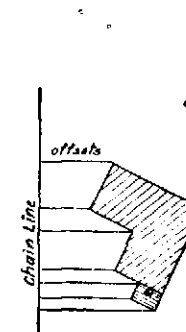


FIG. 15.—Offsets to House.

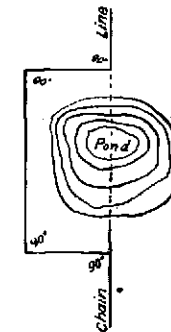


FIG. 16.—Squaring off.

It is thus that the beginner's little difficulties will present themselves to him, and he can best judge how to overcome and avoid them. As regards the house "Rose Cottage," which by means of the chain lines has been plotted in the boundaries, the house and outbuildings themselves are best measured with a tape and sketched in the field book with reference to the position of the respective boundaries.

When a house appears as that near line DF, always measure offsets to the corners, and at right angles to the chain line. Then the salient points can be fixed upon, and measure up the house after, as in fig. 15. Some surveyors prefer to produce the gable, but this, in the author's opinion, is a mistake, as it is useless to suppose it possible to chain lines of indefinite length without meeting obstacles.

When using a chain only we have, to "square off" as in fig. 16, setting right angles as before described. When we come to a wide river it is best to use a theodolite or other angular instrument. These details will be described later. Again, it should be remembered that the ground will not as a rule be level. Where the rise is only small the error is negligible. But all maps and surveys are horizontal projection. Therefore we must hold the chain horizontally, and on steep slopes this must be done in steps, as in fig. 17, using a short plumb line, and using the chain in short lengths. Thus, assuming we have a line 300 links long on the slope. The ground slopes at 1 in

10, $\therefore \frac{800}{10} = 80$ links. Then, by Euclid's well-known proposition that the hypotenuse of a right-angled triangle squared equals the sum of the other

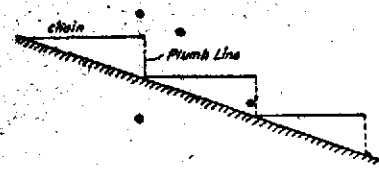


FIG. 17.—Chaining on Slope.

necessity for taking plenty of time and carrying out observations accurately in the field is apparent, because if this has not been done we find something omitted or doubtful, and perhaps have to go all over the ground again to find our mistake out. It is always advisable, in the author's opinion, to do only just as much field work in one day as can be plotted the next. By this means a better idea of what is being done is secured. For plotting we want

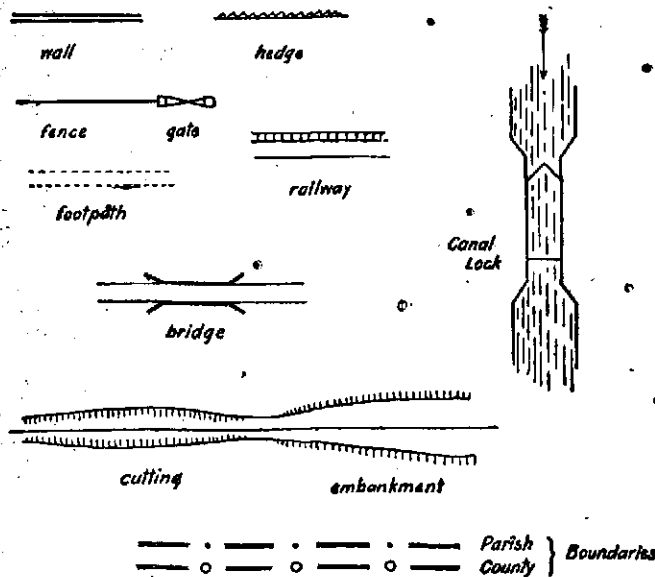


FIG. 18.—Conventional Signs.

a good drawing-board, double elephant size, a few good drawing instruments, set squares, plotting and offset scales, and a rolling parallel ruler.

All these should be of the very best quality, cheap instruments being worse than useless, while good ones last a lifetime with reasonable care. "HH" pencils should be used, and the paper should be damped and stretched on the board, and, for a big survey, glued down. Care should be taken to start with the north of the survey, and to plot it north side up. The main chain lines should then be drawn in on pencil, and should check on with the lines taken. If they do not, something is wrong, and what it is must be found out.

Assuming they are correct, ink them on in faint blue lines. The scale upon which to plot the survey depends upon circumstances. For a small survey 1 chain to 1 inch is suitable. Good scales of ivory or boxwood should be in the possession of every engineer and pupil. They are usually supplied in sets, specially for surveys. Apart from these, for scale-drawings, which as a rule are differently divided, the surveyor should have a 12-in. plotting scale and short offset scales about $1\frac{1}{2}$ ins. to 2 ins. long.

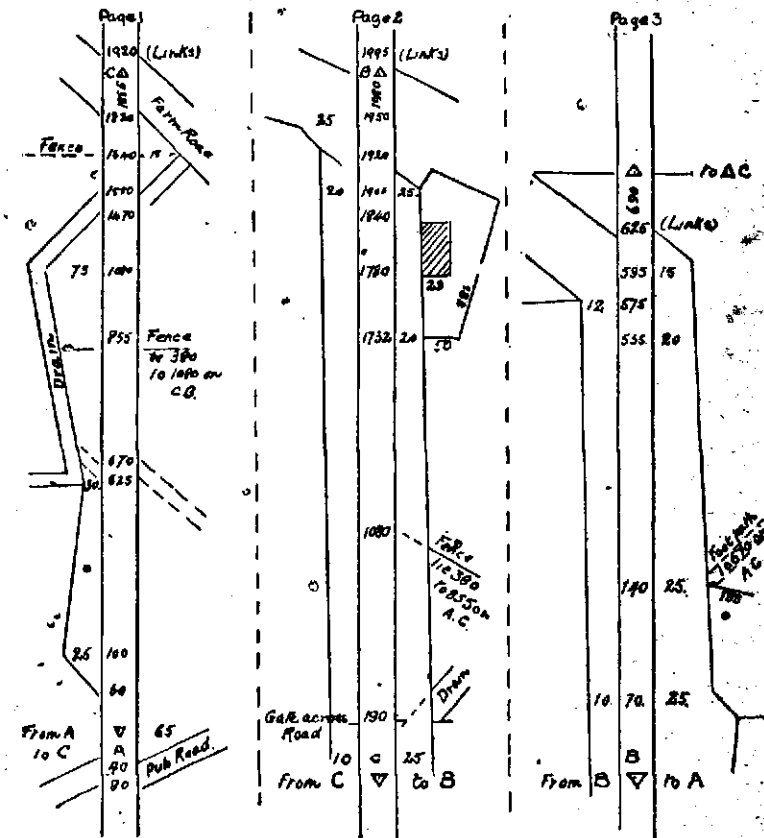


FIG. 18A.—Field Book. Congested Dist. Board Exam., 1909 (Ireland).

The object of these is that the plotting scale may be laid along the chain line and the distances are pricked off thereon, while the offset scales are slid up and down the plotting one, and the offset pricked off at the same time after the manner of tee square and set squares. The first thing to do on any chain survey plotting, or in fact any drawing, is to draw a margin about $1\frac{1}{2}$ in. wide and put on the scale to which the drawing is made, because paper shrinks and expands with changes of temperature and atmosphere, and the scale will therefore vary with the drawing. Always put a north pointer on every map. The making of clean, business-like drawings of surveys is the result of practice.

Good printing is essential. Few, however, can do this at first, and it should be practised. Bad printing ruins a plan. There are also various conventional signs adopted by engineers for map work, of which fig. 18 gives examples.

It is often an advantage to plot maps to the same scale as the Ordnance maps. A scale of links and feet always appears on the latter. The shade lines referred to before are also an advantage, and take very little extra time. In towns, however, and on large and important surveys, and in what is termed

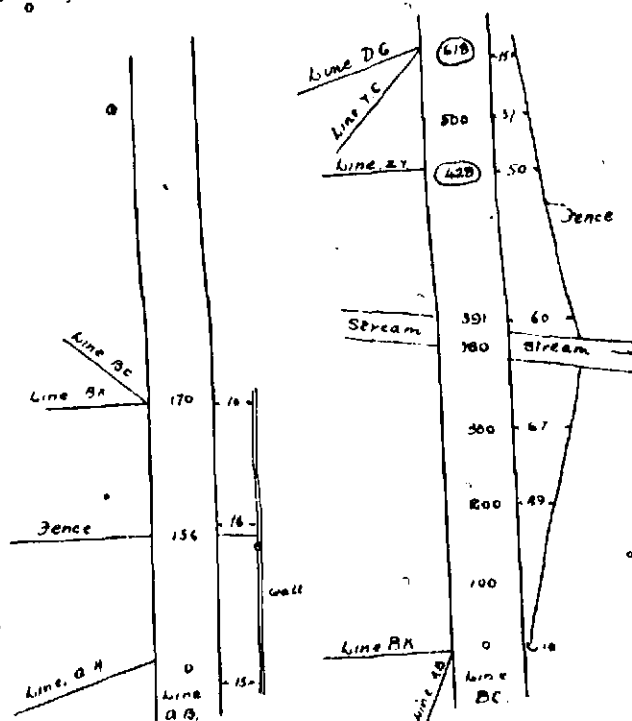


FIG. 18B.—Example of Field Book.

“traversing,” a theodolite has to be used, and the method of using this in valuable instrument will be considered in the next chapter.

For the benefit of the reader I have added an example of a field book which was part of a question set at a recent exam. of the Congested District Board for Ireland for surveyors (September 1909). It is a very good example and should be worked out by the reader for practice, because it is one of those questions dear to examiners, which look so very easy at first sight, and are yet decidedly difficult. The question ran as follows:—

Plot to scale 1-2500 the notes given on the accompanying three pages of field book. Ink in the lines and print title “Patrick White’s Plot” in 1/4 inch block capitals; also write in italic writing 1-2500 scale, 25/344 in. to 1 mil (Max. marks, 120.)

CHAPTER III.

SURVEYING WITH ANGULAR INSTRUMENTS.

WHAT has been said in reference to chain surveying applies only to small surveys in open country, where the chain lines can be seen from end to end and can be chained through continuously. The same methods apply to any kind of a survey, large or small; but when a survey of large size is made, especially in towns and congested districts, and very large surveys are carried out on trigonometrical lines, the aid of an instrument with which to measure angles must be sought.

The principal instrument for the work is called the **Theodolite**. It is an expensive part of an engineer’s outfit; and although many use cheap forms of implements, under the names of prismatic compasses, miners’ dials, etc., for really good work and accuracy there is nothing equal to the transit theodolite. Its use can only be learned by practice, and its easy manipulation and real value are only appreciated by those who have had considerable experience in its employment. Fig. 19 shows a good form of instrument made by Messrs Thornton & Company, King Street, Manchester, and its description is as follows:—

ab is the telescope, having a diaphragm at *c*. This is usually a brass ring, having spiders’ webs stretched across it, a glass with fine lines upon it or platinum iridium points, the upper and lower horizontal wires being what are termed stadia hairs, the use of which will be explained later. *aa* are the four capstan screws which hold the diaphragm in place. On the other side of the telescope is a milled screw to move the object-glass in or out by means of a rack and pinion and so focus the telescope. The eyepiece, which is also capable of a sliding adjustment, magnifies the image, which is usually inverted. This will puzzle the beginner at first. He soon gets used to it, however. The telescope can be so made as to give an erect image, but this detracts from the power of the lenses. A spirit-level *e* is fixed on the top of the instrument, fixed by capstan screws. *g* is the vertical circle fixed to the horizontal axis of the instrument, while *hh* are verniers which should read zero when the collimation line is horizontal. They are read by microscopes. *ll* are the supports, and *m* clips to fix the telescope. *Q* is the vertical tangent screw, while *P* clamps the verniers so as to allow of the final adjustment being made by the tangent screws. The telescope should be capable of being moved through a complete circle. In a transit instrument *rr* are adjusting screws by means of which the horizontal axis of the telescope can be made truly perpendicular to the vertical one. *S* is the upper or vernier plate rotating on the vertical axis *T*. *f* carries the supports *ll*. On this is a compass to take magnetic bearings, and also two spirit-levels *u* at right angles to each other, and also

fixed by capstan screws. Then there are the verniers W and microscope V. The edge of the lower plate (which also rotates) is divided into 360°. Theodolites are known as 5-in., 6-in., etc., according to size of plates. For ordinary work a 5-in. instrument is used, reading to single minutes, $\frac{1}{2}$ minutes or 30 seconds, and $\frac{1}{4}$ minutes = 20 seconds. The two plates called the limbs are capable of being clamped to allow of the use of the slow-motion tangent screw s, and the lower plate is clamped to the vertical axis by the

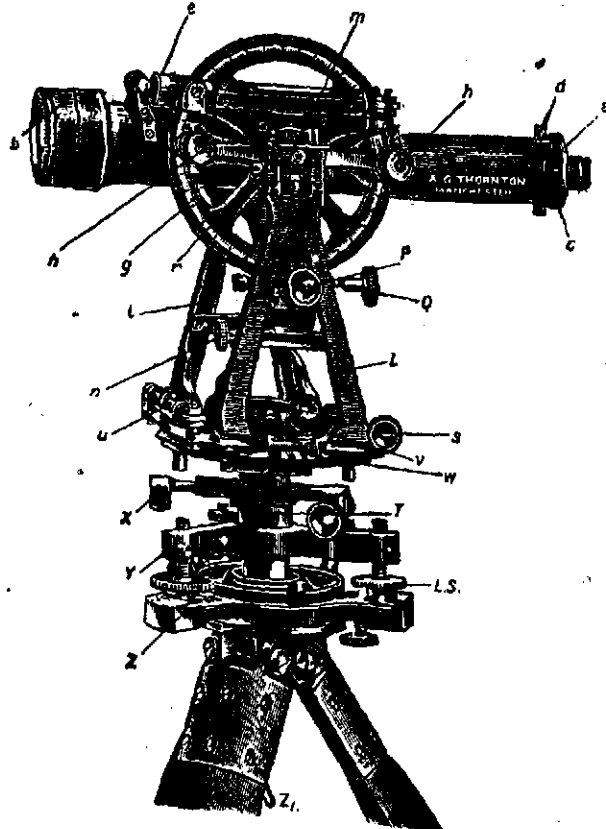


FIG. 19.

screw X. Y is the upper parallel plate, passing through which are three or four levelling screws LS resting on the lower parallel plate Z. By means of these screws the instrument is levelled up, that is, the two limbs are set truly horizontal to the vertical axis on screws truly vertical. Three levelling screws are better than the four with which the older instruments were provided, because four are liable to stick, and necessitate the use of both hands. Finally, just appearing in the figure is a small hook Z₁; the hook should be set in the exact centre of the instrument, that is, in the centre of the vertical axis. It is to the hook that the plumb line is attached when setting the theodolite over a station. In addition to the example given, there are

various forms of theodolite, some having special features of their own, such as Haskold's device for sighting down tunnel shafts.

For further information the reader may refer to any book dealing specially with surveying such as Whitelaw's or Heather's *Mathematical Instruments*.

The next matter to be explained is the use of what are called the verniers of a theodolite; and as their manipulation is often a source of difficulty to the beginner, it will be explained somewhat fully. There are usually two and sometimes three on the lower plate, and also two at h on the vertical circle (fig. 19). They are to enable single and even $\frac{1}{2}$ or $\frac{1}{4}$ minutes to be read off. The actual construction is shown in figs. 21 and 22.

Fig. 21 shows a vernier such as is usually adopted, reading to 1 minute. In this case the length of the vernier is equal to $14^{\circ} 30'$ of the graduated

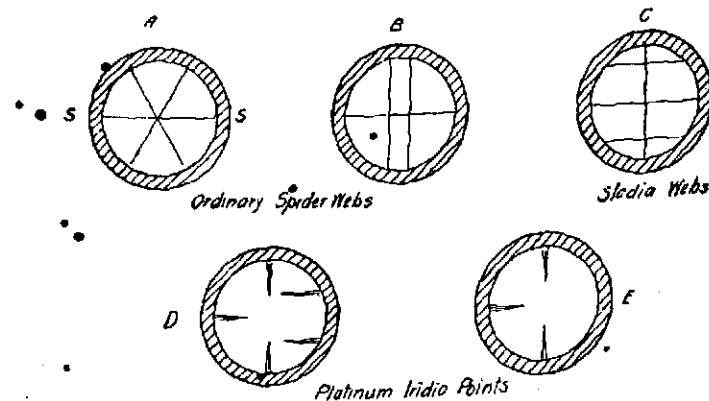


FIG. 20.—Diaphragms for theodolites.

lower limb. This space is divided into 30 equal parts. Therefore each division of the vernier is equal to

$$\frac{14^{\circ} 30'}{30} = \frac{870'}{30} = 29 \text{ minutes.}$$

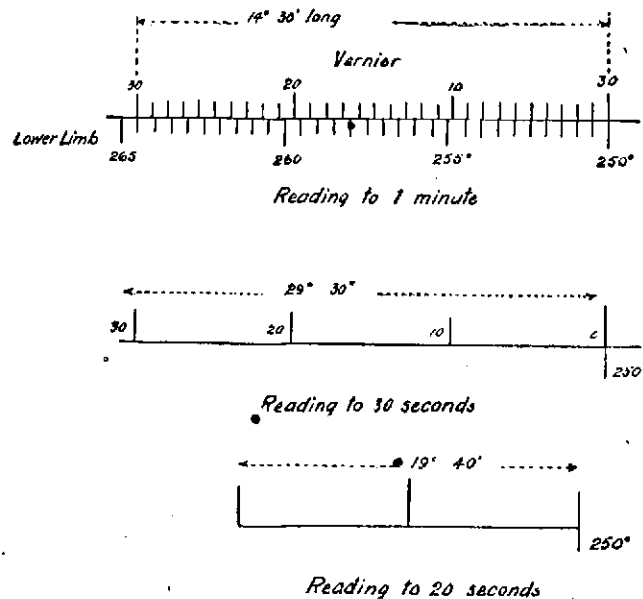
If the zero of the vernier be set to the zero on the lower plate we find that because there is a space of 29 half degrees divided into 30 parts, the first division of the vernier lags behind the corresponding division on the lower plate by an amount = $\frac{1}{30}$ of a half degree, that is, 1 minute, there being 60 minutes in a degree. The second division is therefore behind to the extent of 2 minutes, and so on till the last is 30 minutes behind, and marks $14^{\circ} 30'$ instead of 15 degrees.

Therefore, to set off a desired number of degrees and minutes, set the zero of the vernier to the given numbers of degrees required on the lower plate, and then by means of the tangent screw bring the division of the vernier corresponding to the given number of minutes to coincide with the next division which happens to be in advance of it on the lower limb.

To take an example. Assume we require to set off $14^{\circ} 13'$, the zero of the vernier should be set to 14° , and the position of division 13 on the vernier should then be noted. Then, having clamped the two plates, bring the division 13 to coincide with the division in advance of it. We then have $14^{\circ} 13'$ set off.

The reading of a given angle is of course only the converse process, that is, the zero of the vernier is observed and the number of degrees below it found. This is the number which is nearest it. The division on the vernier which corresponds with one on the lower plate is then sought, and this gives the number of minutes required. Care must be exercised in reading the actual degrees where the divisions do not correspond, because, assuming the reading to be 29° and $30'$, it is the 29, or the *lesser of the two*, which must be read off, because the vernier gives the odd fraction in minutes, which is indeed the purpose for which it is employed.

Naturally the minuteness of the angle which it is possible to read depends on the length of the vernier and also on the graduation of the lower limb. Some verniers are divided into $\frac{1}{2}$ minutes, in which case they have 60 divisions



FIGS. 21, 21A, 22.—Verniers.

of $29\frac{1}{2}'$, and it is then possible to read to 30 seconds or $\frac{1}{2}$ a minute, the length of the whole vernier being $29^\circ 30'$ (fig. 21A). The method of setting off and reading is, however, slightly different.

Assume we require to set off $51^\circ 41' 30''$. Here the number of minutes is more than 30, so we set off $51\frac{1}{2}^\circ$ on the lower plate, and bring up the division of the vernier corresponding to $41' 30''$ minus $30' = 11' 30''$, to coincide with the one in advance of it. Where, however, the number of minutes and seconds is less than 30 minutes, no such subtraction is necessary, the angle being set off direct.

There still remains a third kind of vernier—that illustrated in fig. 22. It reads to 20 seconds or $\frac{1}{3}$ of a minute, the lower limb being divided in thirds of a degree. The length of the vernier is $19^\circ 40'$, and this is divided into 60 parts and read to 20 seconds. The setting off and reading is the same as that for 30 seconds.

The vernier itself is simple enough and gives great accuracy. Its use, however, requires a little practice to understand. Pupils in engineers' offices and other beginners for whom this work is specially intended should read the vernier in the office during any spare time they have on hand. It is also advisable to read both verniers, so as to counteract errors of graduation, and also as a check, but the compass bearing should likewise always be read as well, as it provides a most valuable check on the results.

The practical use of the theodolite can now be explained. Suppose in fig. 13 that we were able to measure the lines BC and BK direct, but that an obstacle prevented us from measuring CK, and prevented the plotting of the triangle BKC unless the included angle CBK could be obtained. It is here that the theodolite is brought into play. It is, of course, assumed that the reader is acquainted with the common properties of triangles and other plane figures, because without such knowledge theodolite surveys would be impossible. The principal properties of plane triangles most frequently made use of in theodolite surveying are—

- i. The three angles of any triangle are together equal to 180 degrees.
- ii. The angles at the base of an isosceles triangle are equal.
- iii. An exterior angle of any triangle is equal to the sum of the two interior opposite angles.
- iv. All the angles of an equilateral triangle are angles of 60 degrees.
- v. In a right-angled triangle the hypotenuse squared equals the sum of the squares of the other two sides.

Reverting to fig. 13, we require to measure the angle at B.

Set the theodolite up as nearly level as possible over the hole left by the ranging pole at B. Get the plumb line and suspend it from the hook Z_1 (fig. 19), steady it, and notice if the bob falls into the hole. This will necessitate a little adjustment of the instrument, then put the plumb bob in your pocket. Now level up by turning the telescope over two levelling screws, and adjust it by the spirit-level lying in the direction across the two screws. Then shift over the other two and do the same. Repeat the process again, and when the bubbles of the levels are in the centre of the run the levelling up is complete, or, in other words, the theodolite will "traverse." Secondly, look at the compass and allow it to settle, then turn the whole instrument until zero on the compass ring graduation is opposite the north end of the needle. Having made sure this is so, clamp the screw X. Then, if this has not already been done, set the plates to zero, and carefully direct the cross hairs on the pole at C. Clamp the plates and complete the adjustment by means of the tangent screw s (fig. 19). Note the compass reading, and see that the instrument has not gone off level. If it has, readjust it. Then unclamp the plate and sight on to K. Clamp again and use the tangent screws. Observe the compass and note down how many degrees the needle has moved through. Then read off the angle by means of the microscopes and verniers. This should then correspond with the compass reading. The next step is to shift the instrument to C, keeping all the plates clamped. Level up and sight on to B, keeping the plates clamped again. Allow the compass to settle. It should read as before, looking in the direction BC. Finally, measure the angles DCX and XCE, and so on, wherever angles are required. They should then be entered in the field book as in fig. 23.

Always take plenty of angles. When coming back to a previous station, take the bearing and note how it agrees with the back bearing. In other

words, when setting up at B, take the magnetic bearing of BA, and when coming back to A, the bearing AB should be the same $+180^\circ$.

This is, however, rather a different subject to merely measuring angle, and is what is known as **traversing**. In the survey illustrated in fig. 13 the lines would be chained in exactly the same way, and finally the angles measured. Then the lines KE, KA, FH, and HL could be omitted, but a much larger amount of chaining of long tie lines is saved in a larger survey.

Plotting Angles.—For plotting, a protractor is used. Protractors are made in various sizes, but the larger the better. One 2'-0 in diameter is a convenient size, as the small sizes are mere toys, although often employed. Lay the protractor on the paper fixed to the board, and mark off a complete circle, that is, 360 degrees, as in fig. 24. It is inside this circle, as far as possible, the survey should be plotted. On a good protractor will be found small circles near the centre as in fig. 24.

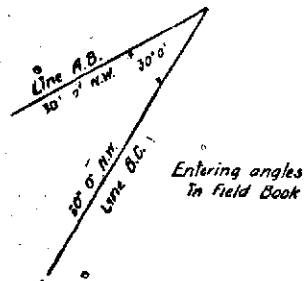


FIG. 23.

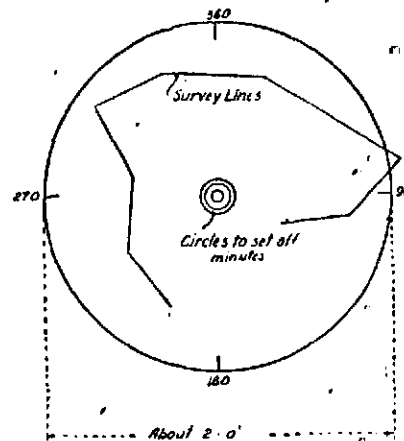


FIG. 24.—Plotting Survey.

Find the centre on the paper. There will be a scratch on the protractor to designate it. Transfer these circles on to it. Then procure a rolling parallel ruler and find what the bearing of the line is; assume it to be $350^\circ 45'$, set one end of the ruler to 350° on the paper and the other on the centre dot. This gives 350° along its edge. We note, however, the three circles. Each of these has value of 20 minutes, then, between 30 and 60, about halfway, is our 45 minutes. Then, keeping the end of the ruler or the circle steady, gently bring the other end downwards to 45 minutes. Roll back to the centre, and we shall see how we have more than 350° , that is, 45 minutes more. This must always be repeated, because sometimes we go to the wrong side of the centre and take less than the required amount. Then roll the ruler to a convenient point and draw the line. Roll the ruler back to see that no error has been made and measure off the length. Treat all the lines in the same way.

Perhaps it may be considered a little premature to treat on magnetic bearings here, but the mere measuring of angles on a survey is not very satisfactory, and magnetic bearings should be taken. These will be explained

under the head of Traversing. If, however, it be required to set out an angle on the paper, only a small protractor may be used.

The real value of a theodolite is on what are known as **traverse surveys**.

These may be either *closed* or *unclosed*. The latter is the method of surveying for a line of inland communication, especially in new countries, and is often of a preliminary nature only, and carried out by means of a miner's dial or prismatic compass. The real nature of theodolite work which concerns

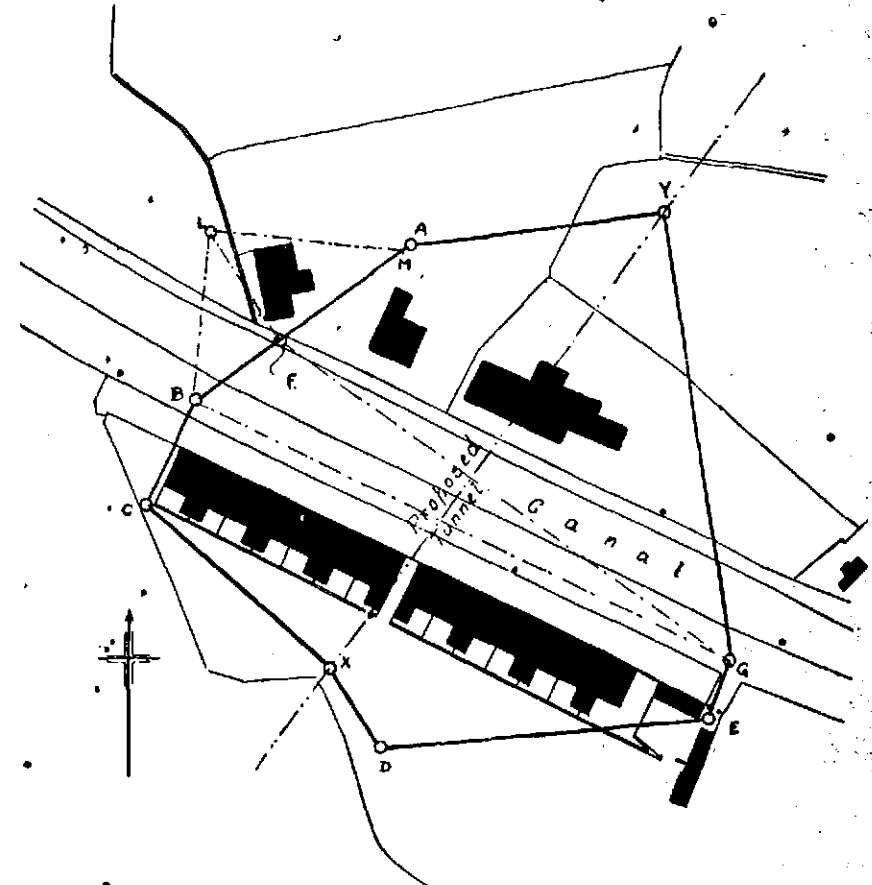


FIG. 25.

the reader is town surveying on the closed traverse principle by the aid of magnetic bearings. It is also used to a very great extent in railway work for setting out curves, in river surveys, and especially for setting out surface and underground lines in tunnel work, which will be treated so far as it concerns sewers made in tunnel and waterworks culverts.

The following is an example of a small closed traverse in a big city.

We shall assume that it is required to make an exact survey of the block of buildings, etc., fig. 25. It is obviously impossible to take any check

lines with which to plot a series of triangles. Therefore the only way is to make a closed traverse by means of the theodolite. It will also be assumed that the lines have all been chained in the usual way, and that all offsets to buildings have been measured, as explained in Chapter I.

In this survey it has been supposed that, although we have a map, it is not correct enough for our purpose, because between the points X and Y it is desired to lay a tunnel under the canal to take a large sewer. It is therefore necessary that the work should be particularly accurate.

As regards the chaining, starting at Y, which is a known point, such as a manhole, we chain on to A, and carefully set pegs at these points. From A to B we have, however, to chain across the canal. Then, having fixed B, when we come to the point F on the canal we proceed as in fig. 25. At F, by means of the theodolite, set off LF at right angles to FA. This being accurate work, the theodolite should be used in preference to other methods. At L sight on to B, and make $\angle BLM = 90^\circ$ to right angle, and note very carefully where it cuts the line FA. Then very carefully measure FL and FM. To calculate the distance FB we have now the very simple equation

$$FB = \frac{FL^2}{FM} \quad (1)$$

= say $\frac{110^2}{130} = 93.077$ links. We can then proceed with our chaining till we come to G, where the same process of measuring angles has to be repeated. There are various problems relating to crossing obstacles by means of the theodolite, but the one in question is perhaps one of the simplest. In the case in point, however, we were fortunate enough to just clear the building which intervened.

In fact, the actual calculation of such distances is often far from a simple matter on the ground, though on paper it may appear so. If, of course, it were not possible to get continuous chain lines, it might become necessary to go a very roundabout way, or even sight through a house if the doorways would permit. In fact, in such work there is ample scope for one's wits, but the most valuable assistance of all is experience.

Chaining across a River, etc.—All the lines being correctly chained and pegs carefully set, we put the theodolite over the point Y and level up. Set the plates to zero and clamp. Allow the compass needle to settle and set it to zero, and clamp the whole system. Now adjust the cross hairs on A, fig. 25, and read the angle turned through, that is, the magnetic bearing. Note it down along the line in question. In the same way take the back bearing YG. This is a check on closing in. As a matter of fact this should be done first, because having read the bearing YA, we clamp the plates and shift to A. Turn the whole instrument and sight the telescope back on Y, and turn through 180° , *vertically*,* and that is so that the telescope points in the direction of Y to A again.

The bearing should now be the same as before.

Now turn on to B and proceed right round till we come to G. Then take the bearings GF and GB also as a help in plotting. They afford an excellent check, and in fact this line BG might well be chained. Finally, take the bearing AY, and see if it differs from the bearing YA by 180° . It should do so, but probably will not. In fact we have what is called a *closing error*, the manner of adjusting which will now be explained.

* That is, on the horizontal axis.

It may first be stated that as a further check on the survey when at F and B we could also have taken the bearing of FG and BG. In fact it is impossible to take too many check bearings or measurements in surveys.

As a further proof of accuracy we can apply the fact that any rectilinear figure contains twice as many right angles, less four, as the figure has sides. The figure in our survey has eight sides, and $8 \times 2 \times 90 - 4 \times 90 = 1440 - 360 = 1080$ degrees, that is, the sum of all the interior angles should be equal to 1080° . If not, something must be wrong. In fact a "closing error" must have been made, and this should be divided equally amongst the sides of the polygon.

There are various rules for this, but the use of one's own judgment is the best; in fact, if the survey has been properly carried out, the closing error should be very small.

As an example of an unclosed traverse survey, take the strip of country in fig. 27 which has to be accurately surveyed.

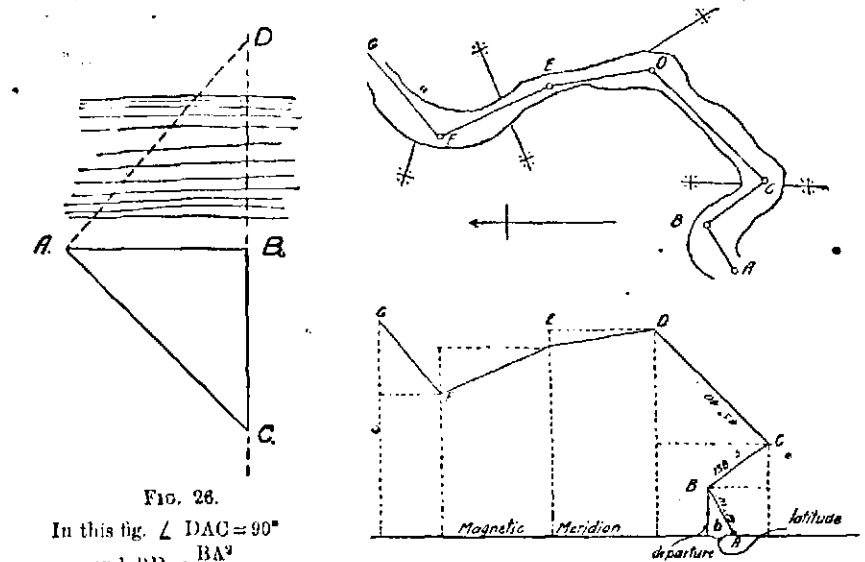
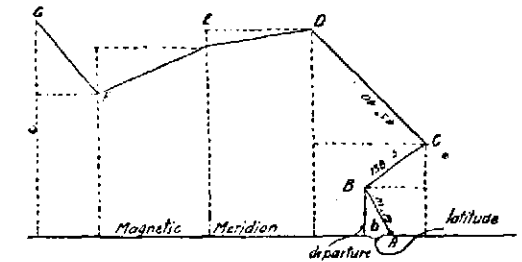


FIG. 26.

In this fig. $\angle DAC = 90^\circ$

$$\text{and } BD = \frac{BA^2}{BC}$$



FIGS. 27 and 28.—Plotting Unclosed Traverse.

The longest possible chain lines are taken as shown and measured carefully, pegs being driven in at the stations, and the offsets, etc., taken. We then set up the theodolite at A in the usual way, and set all marks to zero with the compass turned north. We then take the bearing AB, shift to B, take the back bearing BA, then the forward bearing BC, and so on.

Having done all this, we return to the office and first of all lay down through the point A a line going truly northwards. This line is called the magnetic meridian.

Then as the bearing of AB is known, we can plot it and measure off its length. We have, however, no check on the survey, which therefore requires great care; and plotting such surveys by the protractor alone may lead to error, to avoid which traverses tables have been calculated on the principle of latitude and departure.

Thus assume the chain line to have been 1000 links = AB (fig. 28). At

B set up a perpendicular to the chain line Bb . Then Ab is what is called the latitude and Bb the departure, and knowing the bearing at a (say $61^\circ 10'$), we have the following values of Ab and Bb . That is

$$Ab = AB \cos \angle BAa \quad (2)$$

and

$$Bb = AB \sin \angle BAa \quad (3)$$

It is of course assumed that the reader is conversant with the use of tables of angular functions, as such a repetition of calculation would be very laborious. To avoid this, traverse tables have been prepared, which give for any length of chain line and any magnetic bearing the latitude and departure. By using the tee squares and set squares in place of the parallel ruler we can lay off our first latitude and departure, and plot the actual line without the trouble of plotting the angle, an operation always liable to be somewhat inaccurate.

There are various traverse tables, such as Boileau's, Crellin's, and Gurden's; those who possess Chambers' *Mathematical Tables* will find one therein also, which will do for all practical purposes. To illustrate them take page 376 of the last mentioned tables, where we find that for a distance of 300 links and an angle of 27° we have a latitude 267.3 links long and a departure 136.2 links. By plotting these values we can lay off our line.

If the number is more than 300, or say 476, we add the values corresponding to 300 and to 176, and so on. Of course, in very important work, such for instance as the tunnel previously under consideration, if tables are used they must be more complete than those in the example given. As a certain check on the plotting add up the departures to the east, which should have a + sign entered in the field book before them, and also those to the west. Subtract them, and the total distance of the last line, that is, the departure of the last point, should be equal to the algebraic sum of the departures.

This is an important branch of the use of the theodolite in surveying, and is the one most likely to confront the pupil or assistant in an engineer's office in general practice. Of course there are very many uses to which a theodolite can be put, such as measuring vertical angles, surveying and levelling on slopes, etc.; but as the present work is not intended to be an exhaustive treatise on the subject, but only to explain in as concise a manner as possible the more everyday branches of the work, we will refer the reader to standard works on the subject, a list of which is given at the end of this chapter, if he desires further information as to the other use of the theodolite.

The mode of application in setting out tunnel shafts may now be described. Although we are not concerned here with railway work, in large towns nowadays sewers and culverts very often have to be laid in tunnels; and although such work requires perhaps more accuracy than any other, and is usually carried out by men who have had much experience in such work, a description of its principles will not be out of place here. It was stated that the object of the traverse in fig. 25 was to make a very accurate representation of the ground in order to get the exact position of the two points X and Y , and to plot a tunnel under the canal for the accommodation of a sewer.

Having made certain by careful checking of the survey that the direction of the line XY is truly determined, we direct that shafts shall be driven at X and Y in a manner to be explained later. It is then necessary to transfer the

centre line down below, and there are various methods of doing this. The following, however—introduced, the writer believes, by Mr Simms (whose book on tunnelling may be considered a standard authority on the subject)—is perhaps the most accurate. Referring to fig. 29, two pegs, A and B , are very carefully ranged in the centre line, at such distances from the shaft as to be safe from disturbance. To mark the exact position of the centre line each stake has a spike driven into its head, each such spike having an eye through its top. The eye is then very carefully ranged in the exact centre line, being made visible to the observer by holding white paper behind it. A cord is then stretched through the holes in the spikes to mark the course of the centre line across the mouth of the shaft, and at each side of the shaft a plank is laid at right angles to the string, with its edge overhanging

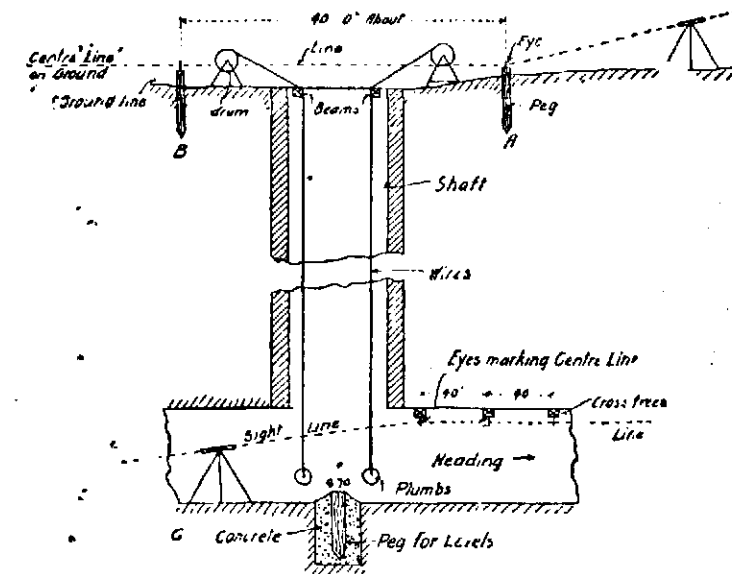


FIG. 29.—Alignment of a Tunnel.

the edge of the shaft a few inches. Two plumb lines are then hung from the planks directly under the cord that marks the centre line. These two wires have plumb bobs at the lower end; these should rest in buckets of water to steady them; then having made sure that the lines are anchored securely in the exact positions (special windlasses, having slow-motion screws to adjust the line to right or left, being usually provided), the engineer takes his theodolite and proceeds down the shaft by means of a cage or ladder. While doing so he must carefully observe every part of the two lines to see that they contain no kinks. When the bottom is reached the theodolite is set up at C in such a way that one wire appears directly behind the other, which is to say, the line of collimation should cut the two wires. In such a position we can range out the centre line below ground.

The wires, and also the cross wires of the telescope, have to be illuminated by lamps, and the instrument should be provided with a special sliding adjustment in order to get into line with the wires exactly. When about

40 ft. of the heading have been driven in the direction of the arrow, blocks of wood having eyed spikes similar to those previously described are fixed to the crosstrees, these spikes being accurately lined in by the theodolite. A cord stretched between them gives the centre line to the underground foreman, the engineers checking it from time to time by the theodolite. The setting out of curves in tunnels will be explained further on. In the meantime we will turn our attention to the setting out of curves above ground.

By reference to fig. 30 we find we have two straight lines DB and BC, which we require to connect by a curve having a radius of 1320 ft. or 20 chains.

Continue chaining the lines till the intersection point B is reached, set up the theodolite at this point, and measure the angle DBC. This is the *intersection angle*. We now require to find out where to start our curve from, or what is termed the *first tangent point*. First we have to find the value of I, the angle at the centre of the curve. It is equal to DBC - 180°. From B measure back towards D, a distance = $T = R \tan \frac{1}{2} I$ (4) R being the radius in chains, then T will also be in chains, and D is then the starting-point of our curve. We have now to set out our curve by what are termed *deflection angles*, on the principle that the angles subtended by any chord at the circumference are equal (*vide* Euclid, iii.). The values of these deflection angles are as follows:—

$$\sin D = \frac{1}{2R} \text{ (chains)} \quad (5)$$

$$\sin D = \frac{d \text{ rad.}}{2r} \quad (6)$$

By means of a table of sines, etc., we find the values of the deflection angles for each chain of the curve. Equation No. 5 is for use with a 66-ft. chain, while No. 6 is for a chain of 100 ft. To show the use of such formulæ in the field we will take an example in practice. As before stated, the proposed radius is 20 chains.

Using Formula 4 we measure back from B the distance equal $T = R \tan \frac{1}{2} I$,

$$= 20 \times \tan \frac{150}{2} = 20 \times \tan 75^\circ$$

$$= 20 \times 3.7321 = 74.6420 \text{ chains} = 7464.2 \text{ links.}$$

Having measured this along both straight as a check, on finishing drive in a peg at D and set the theodolite up over it. Sight on to B and set the vernier to zero.

Now assuming we are using a 66-ft. chain, by using Equation 5 we have the first deflection angle

$$\sin D = \frac{1}{2R} = \sin D = \frac{1}{2 \times 20} = \frac{1}{40} = 0.025.$$

The angle which corresponds to $\sin 0.025 = 1^\circ 25'$. Assuming it, however, to be

a 100-ft. chain, Equation 6 must be used. It has to be worked out by logarithms thus:—

$$\sin D = \frac{d \text{ rad.}}{2r}$$

$$d = \text{chord} = 100 \text{ ft.} \therefore \log d = 2.000$$

$$\log \text{rad.} = 10.000$$

$$\log 2 \text{ radius} = \log 40 \times 100 = \log 4000 = 3.60206.$$

Then

$$\begin{array}{r} 2.000 \\ 10.000 \\ \hline 12.000 \\ 3.60206 \\ \hline 8.39794 \end{array}$$

and the angle which has a log sine = 8.39794 is $1^\circ 26'$.

Still using the 66-ft. chain, however, and having the theodolite set to zero at D, and directed on to B, set off $1^\circ 25'$. Then, with a man holding the chain at D, and another at the other end of the chain having a rod with him, he puts in the rod exactly a chain length from the starting-point and in such a position that the cross wires of the telescope exactly come on to it, always bearing in mind to look at the *bottom* of the rod. Drive in a peg here and then set off $2 \times 1^\circ 26' = 2^\circ 52'$ on the theodolite, and get one of the men to hold the chain at point No. 1, and repeat this process right round the curve. If the curve is at all large, we cannot attempt to do all the setting out from the same point. Assume we shift at point No. 7, set up the theodolite here. Look back at point D, and lay off the deflection angle for the next chain peg. That is the total deflection angle for the peg in question. Then proceed as before round the curve.

There are two other formulæ for deflection angles which may be of use.

They are—

Deflection $\angle$ in degrees =

for one chain

$$\frac{28.65}{R \text{ (chains)}} \quad (7)$$

or in minutes

$$\frac{1719}{R \text{ (chains)}} \quad (8)$$

In tunnel work exactly the same principle is applied, but we must be more exact in our calculations, and use arcs instead of chords. That is to say, the exact deflection angle of any curve

$$= D = \frac{\text{arc}}{2 \text{ radius}} \quad (9)$$

which, however, gives circular measure, which must be multiplied by 57.3 to give degrees, minutes, etc., because there are 57.3 degrees in a radian, the latter being any arc of a circle of a length equal to the radius of that circle. As a check on the setting out of our curve, it should be found when we come

If a whole series of long levels be required at fairly equal distances apart, such as the proposed route of a road, railway, or canal, they are usually taken at every chain. Such a method may be adopted with success by setting up

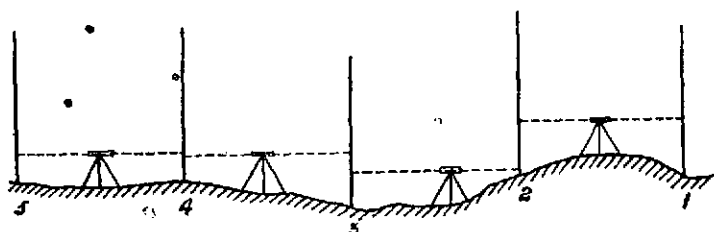


FIG. 37.—Levelling.

midway between any two points, and getting the difference of level between those two points, as in fig. 37.

Such a method is, however, somewhat tedious in short levels, such as those in towns for drainage schemes, where it would be possible to sight two or three points from the same setting up of the level.

A slightly different procedure is adopted in such circumstances. Assume

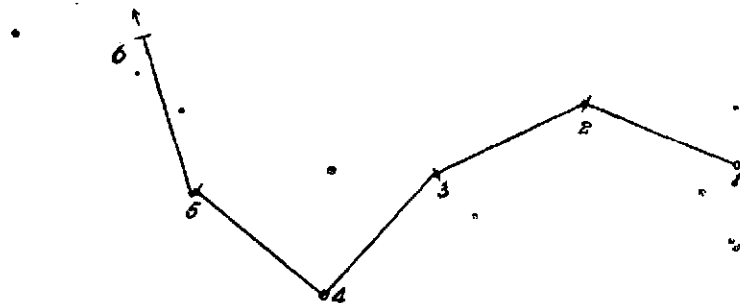


FIG. 38.—Plan of Levels in fig. 37.

there are certain points in fig. 38 that we want to take levels on so as to plot a section, and that we set up the level as described midway between each and ascertain the differences of level. We proceed to book them as follows:—

Back Sight.	Fore Sight.	Rise.	Fall.	Reduced Level.	Distance.	
5.01	7.04	...	2.03	97.97	1-2	41.45
9.81	6.02	3.79	...	101.76	2-3	34.06
7.62	9.73	...	2.11	99.65	3-4	7.39
8.91	4.24	4.67	...	104.32	4-5	11.53
10.10	7.03	3.07	...	107.39	5-6	4.14
						7.39
41.45	34.06	11.53	4.14	...	...	

In this tabulation all the fore sights are booked in one column and all the back-sights in another. Note where there is a *rise* or a *fall*, then subtract the two and put the result down either under the rises or falls.

We then usually have to reduce the level. In this case we have subtracted the first fall, 2.03 from 100.00; to the result, 97.97, we add the next rise and get 101.76, and so on.

By far the most usual method is, however, that of **back, intermediate, and fore sights**. When we set up the level the first sight we take is a **back sight**; all the others are **intermediate** until we decide to shift the level, when the last sight before shifting is called a **fore sight**.

We then proceed as follows:—

1. B.	2. I.	3. F.	4. Falls.	5. Rises.	6. Reduced Level.	7. Distance in Yards.
0.87	9.00	...	8.13	...	50.00	0
...	2.32	...	...	6.68	41.87	19
...	9.96	...	7.64	...	48.55	27
...	9.05	...	...	0.91	40.91	34
...	...	10.97	1.92	...	41.82	59
5.15	...	8.51	3.36	...	39.90	75
...	...	...	...	...	36.54	89
...	...	...	21.06	7.59	13.46	...

The first sight is a back sight and goes in column 1. The second, an intermediate, goes in the second column on the same line. All the others follow downwards until the last before shifting the level. This goes on the third column on the line below.

The next sight with the new position of the level goes in the first column, again with a new line. Starting to make up then from 0.87 to 9.00, being a rise of figures is a fall in the ground, subtract 0.87 from 9.00 and put it in the falls column. The next level is 2.32. This is a rise. Subtract 2.32 from 9.00 and book it in column 5 under rises = 6.68.

This process is continued until the level is shifted, when the making up starts afresh. Having carefully assorted the rises and falls in separate columns 4 and 5, we put at the top of column 6 the distance in feet of our datum below any suitable point, which is usually a bench mark. We have explained the term before, and it is only necessary here to add that any series of levels, where practicable, should be plotted in reference to a bench mark, that is, the first sight should be taken on to such a mark where handy.

Assume the reading to be 8.29. We decide to make our datum (which is the imaginary horizontal line upon which the heights of a section are plotted) 50 ft. below such a mark. Obviously a fall of 10.20 makes the surface of the ground 39.80 above that datum. Or we may, and in fact usually do, take the actual height of bench mark as found at itself or on the map of the district. Looking again at the first line we find 8.13 in the falls column. Then subtracting 8.13 from 50.00 we have at a distance of 19 yards a height of 41.87 above datum. In the next line we find a rise of 6.68. Add this to

to the second tangent point that the total deflection angle, or the deflection angle for the whole curve ($\angle BDC$, fig. 30), should be equal to half the angle I , at least within the reasonable error of a few seconds. To ascertain the exact length of any curve, divide half the intersection angle ($=I$) by the deflection angle for one chain. In the example given assume that

$$I = 150. \quad \frac{150}{2} = 75.$$

$$\therefore \frac{75}{1.25} = 60 \text{ chains length of curve.}$$

The above method of setting out curves is that usually practised by engineers. It is accurate and fairly simple, because there are tables to be had giving values of deflection angles for various curves. It is also accurate

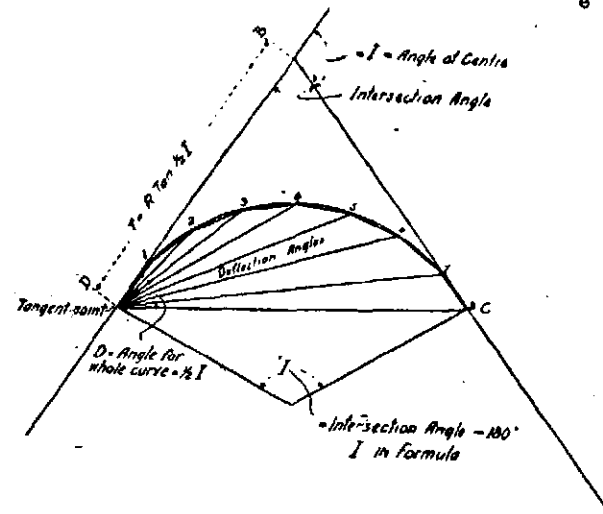


FIG. 30.—Curves by Theodolite.

on moderate slopes. There are other methods, for which the reader may again be referred to treatises on surveying. It is also possible to set out curves without a theodolite when the actual curve itself is of not much importance, that is, on a country road or canal, where perhaps a theodolite is not handy, and the work is left to an assistant or an intelligent foreman.

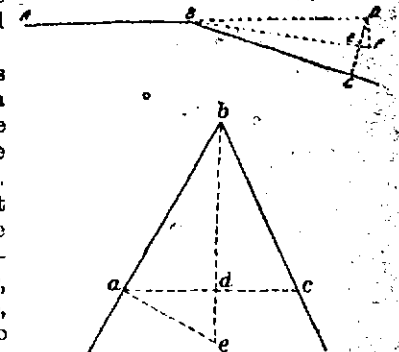
Setting out Curves without a Theodolite.*—The following is a convenient method of ascertaining the tangent points of a circular curve of given radius by construction when the straight lines that are to be connected by a curve are already fixed on the ground, and when the unknown angle contained by the lines is apparently not much less than a right angle. Let AB BC (see fig. 31) be two straight lines to be connected by a circular curve of given radius. Produce AB to D, making BD an aliquot part of the radius. Set off BC equal to BD, and measure the chord DC, bisecting it at E, and at D set off and measure DF at right angles to BD, meeting BE produced at F. Then

* *Building World.*

F will be a fraction of the required tangent of the same amount as BD is the proposed radius.

For example, let the proposed radius be 500 ft., and let BD be 100 ft. (one-fifth of the radius), and suppose that F measures 26.8 ft. The required tangent will be $26.8 \times 5 = 134$ ft.

When the chord DC is 1.4142 times the length of BD or BC (that is also when $F = BD$), the angle of intersection will be right angle, and with a smaller angle the offset DF becomes inconveniently long. It will be better then to find the tangent by similar triangles set out within the angle. Thus (see fig. 32) take any suitable lengths $ab = bc$, measure the chord ac , and bisect it at d and at a or c (say a), set off, and measure ae at right angles to b , meeting bd produced at e . Then the required tangent will equal $\frac{ab \times \text{radius}}{ae}$.



FIGS. 31 and 32.—Curves without Theodolite.

the value of the angle of divergence dbc , or that of intersection abc , is required from these measurements, it is essential that the latter be done very accurately, since the true value is accordant with the pains taken with them.

Before leaving the subject of theodolite surveying, it may be well to explain the elementary use of the stadia wires now usually provided, and as illustrated in fig. 20.

Take, for example, either C or D. It will be noted that on C there are an extra pair of horizontal wires, and on D an extra pair of points. These are always very carefully set a fixed distance apart. Their use is for measuring distances. Suppose we have a long rod divided into feet and 10ths, such as the levelling staff described in Chapter IV. If we look through the telescope we see a certain distance appear on the staff to be between the stadia wires.

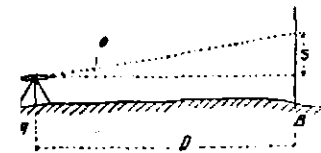


FIG. 33.—Distance by Ordinary Theodolite.

This distance then is a certain fraction of the actual distance from the centre of the instrument to the staff. This fraction is usually $\frac{1}{100}$. That is, if we can see 2.72 ft. on the staff, the distance on the ground is 272 ft. This is useful in measuring long distances or checking them over.

Again, we may use the ordinary theodolite as in fig. 33. Set everything level and sight on to the staff held at B. Note the reading, then turn the telescope vertically a reasonable distance and take another reading. Also note exactly the vertical angle turned through. Then

$$D = S \cot \theta \quad \dots \dots \dots (10)$$

say

$$S = 10 \text{ ft.} \times \theta = 2^\circ.$$

then

$$\cot 2^\circ = 28.636.$$

$$\therefore D = 28.636 \times 10.00$$

$$= 286.6 \text{ ft.}$$

CHAPTER IV. LEVELLING.

LEVELLING is the art of representing on paper a profile of the earth's surface at any point.

By surveying as has been described in the previous chapters, we were able to represent on paper the surface of the earth considered as a plane

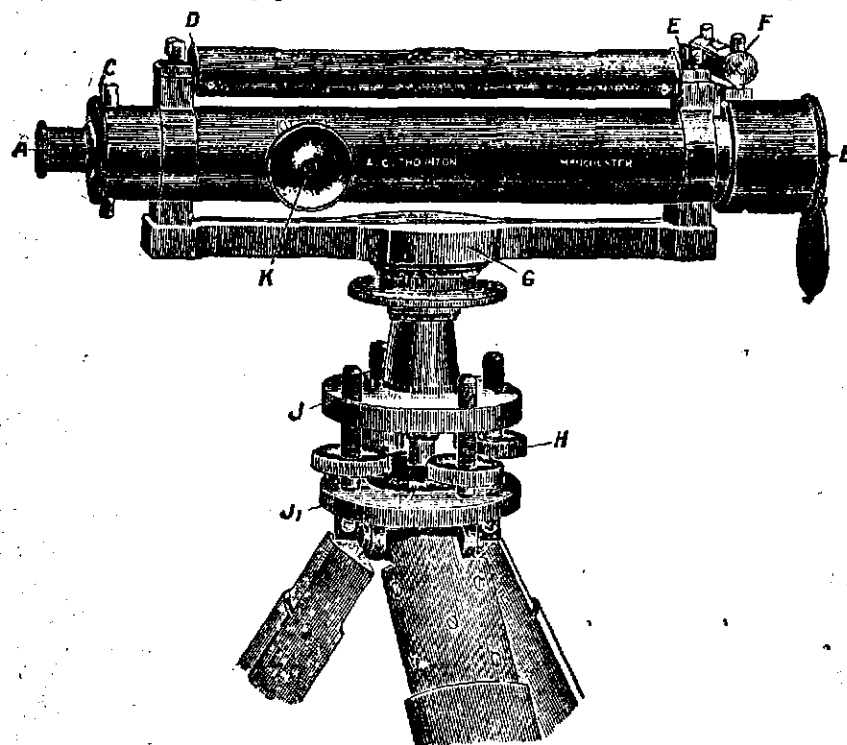


FIG. 34.

surface. That it is seldom really a plane surface is obvious to everyone, and it is to show how far it differs from such a surface that the art of levelling is resorted to.

The work of levelling consists of—

1. Using the level and staff to take the readings.
2. Booking the readings and making up the levels.
3. Plotting the section from the reduced level to a suitable scale.

To start with we must describe the engineers' level in "detail." The one form now mostly used is called the Dumpy Level, and was an invention of a civil engineer of the name of Gravatt.

The usual type is illustrated in fig. 34.

It consists of a telescope AB similar to the theodolite in fig. 19, with a diaphragm at C carrying cross wires similar to B, fig. 20. A spirit-level DE is supported on capstan screws for adjustment and a small cross level F is fixed in a similar way. At G, in most good instruments, is a compass, though in the illustration it has been omitted. Levelling screws are provided at H and parallel plates at J and J1.

The particular instrument illustrated is made on the "entire" principle, that is, the telescope is in one with the parallel plates, although in most instruments it screws into them. At A is the eyepiece, at B the object-glass, and at K the diaphragm screw.

Levelling staffs are of many types, but the simplest is the best, as in fig. 35. This is divided into feet and 10ths, because additional figures and divisions are likely to confound the leveller, and the second decimal place can readily be found with practice. For underground work, however, with short sights a more finely divided staff may with advantage be used.

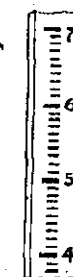


FIG. 35.—Levelling Staff.

N.B.—In most levels the figures appear inverted thus 9.

The simplest form of levelling which presents itself is the difference of level between any two points, such as fig. 36. Here we have two points, A and B, a certain distance apart, and need to ascertain the difference of level between them. Set up the level as near as possible midway between A and B, and level up very carefully. This is more important than when using the theodolite, and the levels are far more sensitive, as the whole operation depends on this adjustment. Now move the eyepiece in or out and see the cross wires clearly, and then turn the pinion screw and bring the staff at B clearly into view. Where the horizontal wire cuts it read off the number, care being taken that the staff holder holds the staff quite vertical between the vertical webs. Some staffs have a small plummet thereon to assist the staff holders.

FIG. 36.—Difference of level between two points.

Shift the staff to A and repeat the process, assume the reading at B to have been 13.02 and at A 12.61, the difference $13.02 - 12.61 = 0.41$ of a foot, or 4.92 inches. As the first reading was greater than the second, the first point is lower than the second. In other words, "a rise in figures means a fall in the ground," and *vice versa*; this must be carefully borne in mind, as when making up levels it is always necessary to know if we have a rise or a fall between two points.

the last sight in column 6. Then at a distance of 27 yds we have a height of 48.55 above datum.

Before making these calculations it is necessary to have some means of ascertaining that there is no mistake in the making up of the levels. Add up columns 4 and 5. These give 21.05 and 7.59 respectively. Now looking down column 6 we find our last reduced level is 36.54; subtract this from the first, that is, 50.00. The difference = 13.46; subtract 7.59 from 21.05 and we have 13.46 again. If, then, through any series of levels, these two calculations agree, the making up is correct. This precaution should on no account ever be omitted.

Having now got the levels correctly taken, booked, and reduced, we draw on the paper our datum line horizontally and decide upon what scale we shall plot our section. This, of course, will depend upon circumstances.

For good working sections, 30 ft. to 1 in. vertical and 300 ft. = 1 in. horizontal, 20 ft. vertical to 300 horizontal and larger ones are suitable, and, within reasonable limits, the larger the better.

For parliamentary work the Ordnance scales are best used for horizontal, the same as the plan, and 100 ft. to 1 in. vertical, or 50 ft., etc., are used (see Parliamentary Standing Orders).

Lay down these two scales, therefore, on the paper. Now carefully on the datum line write its height with reference to some point, or call it Ordnance datum if it is so.

Then carefully prick off the horizontal distances given in column 7. Check them over, or, better still, let someone else do it, as a guard against personal error. From each of these points draw vertical lines. Near the bottom write in the heights of the respective points above Ordnance datum, compare these again with the section book, then prick off the heights on the vertical lines. Finally, join these points.

We have now, on a *distorted* scale, what is necessary to show the irregularities of the ground, a representation of the surface along the line required.

The datum should be in black, the verticals in blue, the surface in black, the figures in black, and all new work drawn on the section afterwards in red.

The following is an actual example taken from practice, and was a section made when drawing up plans and sections for a roadway, bridge, and weir, under the author's supervision. It was a somewhat tedious piece of work, owing to the nature of the ground and the presence of trees and swamps.

First of all are given the levels as taken, booked, and reduced.

From that data, and referring to fig. 39, we draw the datum which is Ordnance datum and the scale, and carefully set off the distances shown in column 7. These have been carefully chained and reduced to the horizontal by methods previously stated. We now draw the verticals and mark the heights, etc., and finally the surface line.

The drawing in fig. 39 has been made so as to make clear how an ordinary section should appear on paper.

All lines appearing in black on a working section are drawn as solid lines, thus: ———

All those appearing in red are marked ———

„ „ blue are marked ———

Proposed embankments are hatched thus  coloured yellow.

„ cuttings, cross hatched, thus  coloured red.

BRAY PROPOSED BRIDGE AND WEIR LEVELS AT COW BANKS, 1.10.08.

1.	2.	3.	4.	5.	6.	7.	8.
Back.	Inter.	Fore.	Falls.	Rises.	Reduced Level.	Distance.	Remarks.
4.31	...	5.23	...	...	74.7	0	O.B.M. at Glen Herbert.
4.34	...	5.23	0.89	...	73.81	40	Centre of footpath. •
6.25	...	7.35	1.10	...	72.71	286	Top of steps.
0.27	...	16.47	16.20	...	56.51	311	On slope.
0.47	...	15.66	15.19	...	41.32	361	On slope.
3.17	...	12.54	9.37	...	31.85	403	On path.
4.18	...	9.50	5.32	...	26.53	460	In field.
3.80	...	3.90	0.10	...	26.53	530	South edge of mill race at water-level.
2.37	1.10	...	...	1.27	27.80	553	Top of bank.
...	6.41	...	5.33	...	22.47	591	In field.
...	9.94	...	3.51	...	18.96	633	South side of river at water-level.
...	...	9.20	...	0.74	19.70	758	North side of river.
7.39	...	3.85	...	3.54	23.24	803	On road.
6.37	...	5.42	...	0.95	24.19	1070	Centre of Lower Dargle Road.
...	...	...	57.01	6.40	...	...	

 Levels taken by F. Noel Taylor.
 Staffman, John Byrne.

Sum of falls	57.01
Sum of rises	6.50
Difference	50.51
First reduced level	74.7
Last	24.19
	50.51

When the actual section is made, we can then see where cutting and bank and other works will be necessary, and can proportion them accordingly. This will be further described under Earthworks.

The heights measured to the proposed surface of the new roadway are scaled off and written in red. The river-level is the mean. On all such works it is necessary to also show the *highest recorded* flood-level where it is attainable. The gradients should also be marked on; they are easily calculated as follows:—

Assume the distance between two points to be 240 ft.

By the section we find the total rise in the ground between these two points is 9 ft.

Therefore $\frac{240}{9} = 26.66$, a gradient of 1 in 26.66.

Or another way sometimes comes useful, especially in answering examination questions.

Say we have a slope of 1°.

Look up the natural cotangent of 1°.

We find it is 57.28.

The gradient is then 1 in 57.28.

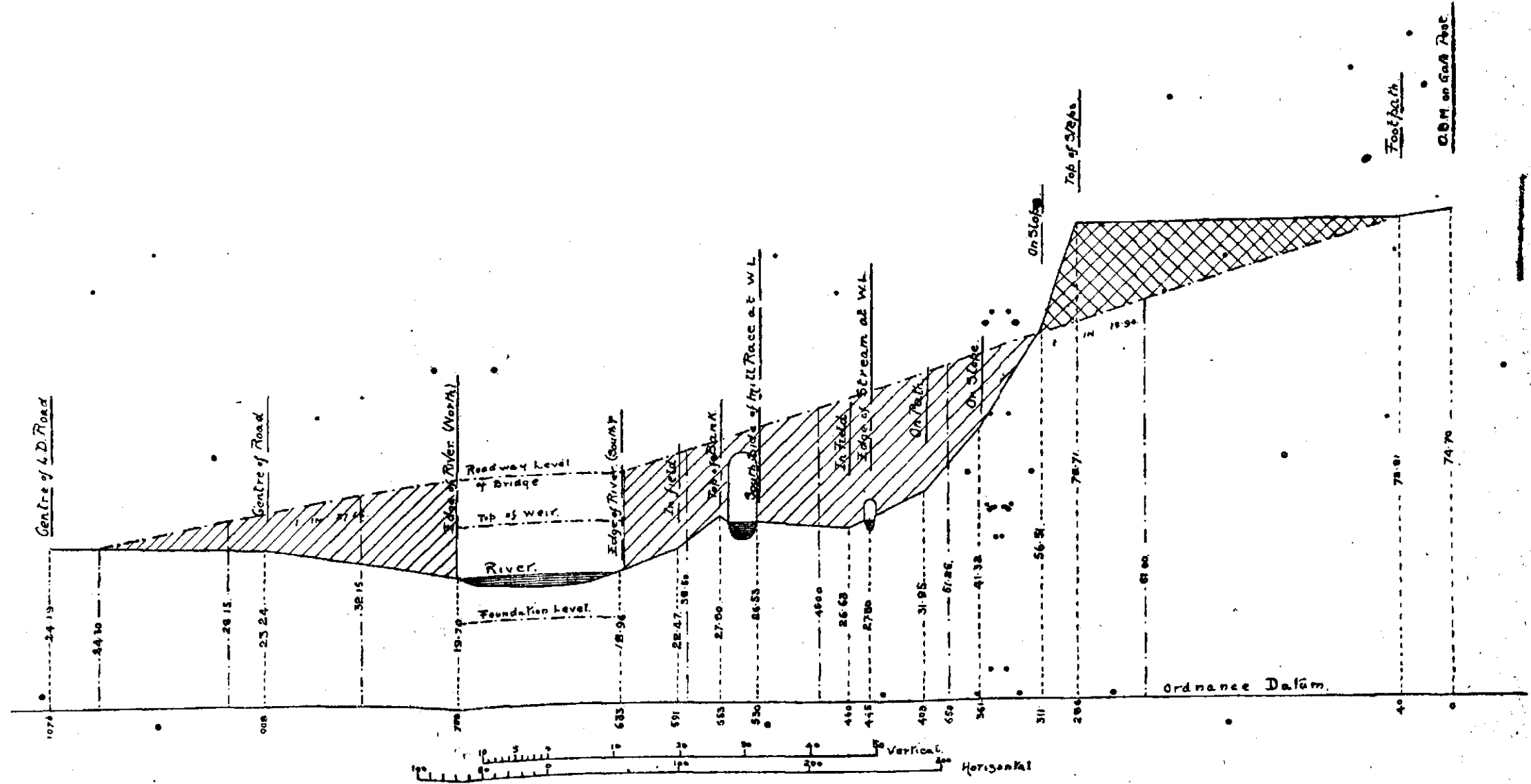


FIG. 39.—Example of Section for an ordinary piece of engineering work.

Now the example given is a typical one likely to confront the reader, and is the sort used for similar works, roads, pipe lines, and drainage schemes.

As regards the latter, we shall give further examples when we come to drainage. Before leaving the subject it would be well to put before the reader an example of a section for railway work. These are done somewhat on a more elaborate style, and have levels at every chain length.

An example is given in fig. 40. Here we have the chainage and miles marked on the datum. The lengths above datum are marked in the usual way, such as 261.35. This appears on the bottom column, and is written in black; above it, in red, appears the height above datum to level of rails.

Above that again is the difference between the two, which may be cutting or embankment.

If cutting, the figures go in red.

If bank, the figures go in blue.

The surface line is black.

The datum line is black.

The formation line, viz. tops of banks or bottoms of cuttings (commonly called the road bed), is red; the level of the rails (sometimes drawn in blue; and on any section the amount of cutting should be coloured red (very light), and the bank yellow (very light).

For levelling on slopes and setting out cuttings

and banks we often use a little instrument called the clinometer. It is especially used by American engineers. It is illustrated in fig. 41, and

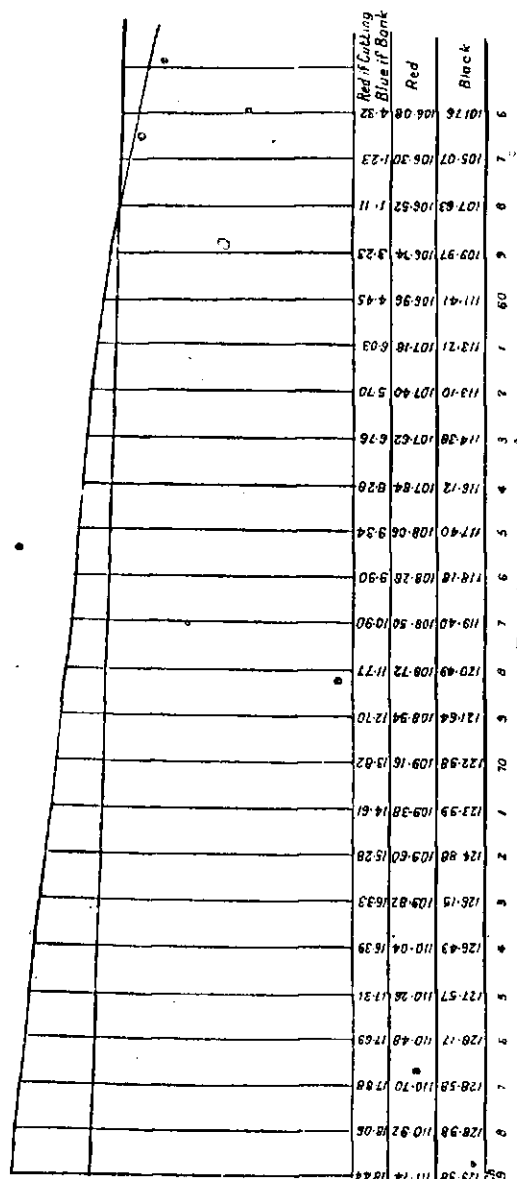


FIG. 40.

consists of a telescope, a vertical circle similar to a theodolite, sometimes a vernier and a microscope; there is also a spirit-level; then by resting it on the top of, for example, a ranging-pole, we can take rough levels by means of the spirit-level provided; we can also set out gradients.

As explained before, the natural cotangent of the angle of slope gives us the gradient; therefore, holding the clinometer on a staff and setting off the angle required by the circle, which is moved by a rack-and-pinion motion, we can set off the gradient.

Example.—Assume a gradient of 1 in 34 is required; 34 is the natural cotangent of $1^{\circ} 41'$; referring to fig. 42, place a pole in at A and rest the clinometer on the top of it. Some instruments are provided with stands, and are very useful.

Set quite level, and then turn the milled screw until it reads $1^{\circ} 41'$. Of course the operator must set the vernier to zero beforehand. Hold a pole at B, not too far away, as the instrument is only a small one.

Note where the cross wire cuts it, then very carefully ascertain the height of the collimation lines of the instrument above the ground; let this be 5 ft., then drive in a peg at B exactly 5 ft. below where we made the mark of sight, then the ground when excavated or filled in in a straight line between bottom of the pole at A (which in fact should rest on a previously set peg), and the peg at B lies on a gradient of 1 in 34. A theodolite may be put to

the same purpose. By using the vertical circle in the same way we can get the angle of slope and gradient of any slope by simply reversing the process. Or we may, instead of the poles, hold the clinometer on a long straight edge.

To the engineering pupil it is most essential to know how to level correctly and quickly, and to book and reduce his levels and plot a section. It is probably the first thing he will be shown. I

have tried to point out the principles

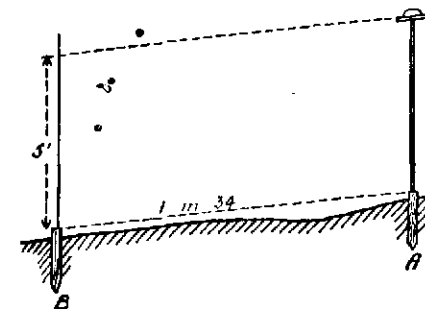


FIG. 42.—Use of Clinometer.

and practice of levelling as concisely as possible in this chapter. To take levels and plot sections competently, however, the reader should in his spare moments endeavour to acquire some practice in the field at reading the staff and booking, and also to get some levels. There are sure to be plenty in his office if he is a pupil, and he should try making them up and reducing them, because the beginner is often astonished to see how little he knows when first entrusted with a piece of levelling work on his own account. Levels can also be taken by an ordinary theodolite by setting the telescope and plates truly level and clamping them. It is also useful, on steep slopes, to set the telescope to the most convenient angle, that is, the one approaching

But we want to set pegs so as we will have the correct gradient, a most important consideration in drainage work, where fall is small, and on roads, railways, and canals.

By referring to the section, we see pegs have been already set at a fixed height above datum. Assume, when all the new works have been put on the section, that a trench 420 yards long with a gradient of 1 in 200 has to be put in. If at the beginning of such a work we have a height above datum at surface of 27.04, and the depth of trench at the point was calculated at 7' 6" = 7.5 ft., then $27.04 - 7.5 = 19.54$, the height above Ordnance datum of the bottom of the trench at that point is 19.54 ft.

Now we previously had at a short distance away (say 120 ft.) set a peg at 23.24 ft. above O.D. and had painted thereon (which it is advisable always to do on such pegs as are frequently referred to) those figures; we set up the level about midway between that peg and the one we want to set. Having directed the foreman to drive in his peg lightly to what he considers a near approximation to the correct level, get someone to hold a staff on the set peg and book the reading. Assume it to be 14.51. Now it is desired to set our new peg to 19.54, and $23.24 - 19.54 = 3.70$ ft. below the set peg. If the last reading (the back sight) was 14.51, a fall in the ground means a rise in the figures on the staff. We add, therefore, 3.70 to 14.51.

$$\begin{array}{r} 14.51 \\ 3.70 \\ \hline 18.21 \end{array}$$

Next, have the staff held on the new peg, and direct that it shall be driven down or raised until the staff reading is exactly 18.21. The peg is then set. But we have 420 yds. of trench at a gradient of 1 in 200. Assume that it can be seen from end to end, although in practice this is not always the case. We then have to set more than 2 pegs. Now $\frac{420}{200} \times 3 = 2.10 \times 3$, that is a rise or fall (say rise here) of 6.30 ft. in 420 yds. Therefore at the other end we want a peg whose height above Ordnance datum is $19.54 + 6.30$, and $19.54 + 6.30 = 25.84$ ft. above O.D. Now reset the level between the two points and read the staff when held on the peg just set. The reading we assume to be 10.30. We now want to set a peg 6.30 above this, which means we must read 6.30 less on the staff, and the peg must therefore be adjusted till the staff reading is 4.00; we then have the two ends of the trench set at the correct levels—although this reverse process of levelling is somewhat confusing at times to the young engineer—and the rest is set out by the engineer or foreman in charge by means of what are termed *boning rods*.

These are illustrated in fig. 45.* Their use is simply to give a straight line in a vertical plane between two fixed points, whether up or down hill or on the level; in other words, they enable a uniform gradient to be kept between the two fixed points. A boning rod is like a rough tee-square used with the blade upright and the cross piece horizontal. Not less than three of identical height are used together, as follows. Pegs having been driven so as to give the level at the two extremities of the section (which may be part of a much longer line), the point of a boning rod is held on each of the pegs,

* *Building World.*

while the third boning rod is held at any intermediate point or points where the level is required. The surveyor or foreman sights through over the crossbar of the boning rod or one of the pegs to that on the other peg, and the man at the intermediate boning rod raises or lowers it, by signal from the surveyor, until the top sights through with the end rods, as shown in the accompanying diagram. The point of the intermediate rod may have to be lowered, the earth being kicked away and flattened, or may have to be raised by putting a brick or two underneath, but small wooden pegs are easily fixed to the required level and are more distinctive. Of course, when giving an example of setting out levels, two pegs were put in 420 yards apart. This, in practice, is ever so much too long a base over which to use the boning rods

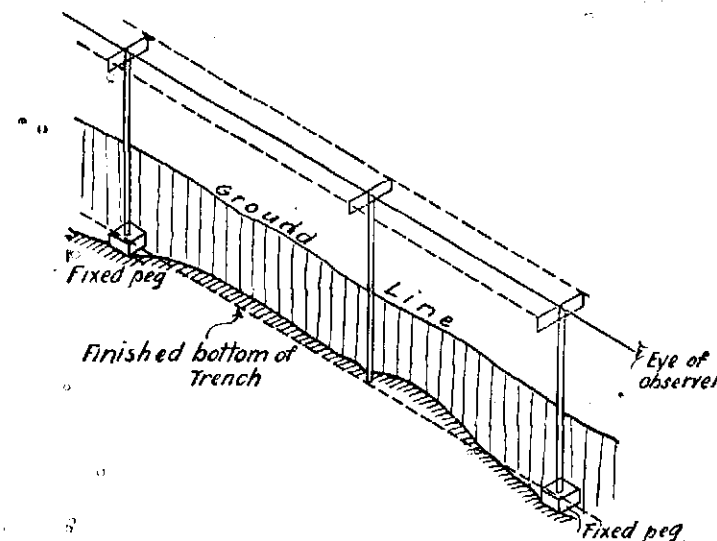


FIG. 45.—Use of Boning Beds.

to ensure accuracy, 100 to 200 ft. being quite far enough apart for this purpose.

Levels underground are carried out in exactly the same way, but of course the work demands too much accuracy to allow of the use of boning rods, all pegs requiring to be carefully set by the level. These pegs, too, are very accurately set in brickwork or concrete, and flushed up with cement, as in fig. 29, and to transfer our surface levels below we proceed as follows.

Near the top of the shaft we put into the brickwork an iron stirrup similar to footsteps for use in manholes. Then by use of the staff we find the exact level of the top of this stirrup. The engineer then gets a steel tape (whose accuracy is known), and then proceeds down the shaft and very carefully measures each tape length till he arrives at the bottom. Then another stirrup is let into the brickwork of the shaft, and its level calculated and marked thereon. Finally, by means of plumb lines and the steel tape, a peg is very carefully set to a suitable level, as described and shown in fig. 29. The whole operation is then checked, and should agree with the original measurements within a small fraction of an inch. From this bench

that of the slope in question; then, having the staff held on the points required to be levelled, set down the reading just as in ordinary levelling.

We, however, have a certain + or - correction for declivity, according as the inclination is up or down from the telescope. This correction is equal to the distance of the point from the axis of the instrument, which should be at the exact foot of the slope multiplied by the sine of the angle at which we have set and clamped the vertical circle.

All the distances are measured on the slope and no correction is made.

Another use to which levelling is put is that of contouring. We have explained the meaning of contouring in Chapter I.

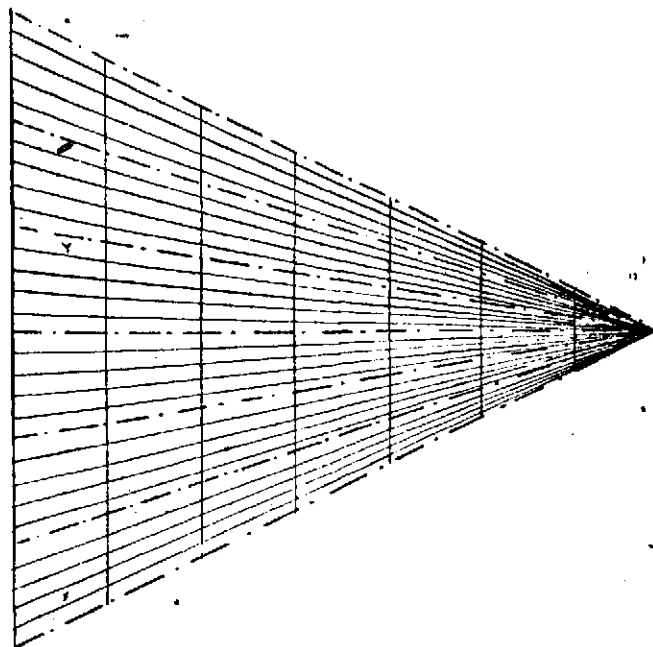


FIG. 43.

To plot a contour we require two horizontal and one vertical co-ordinates. The distance apart varies, and the usual way to decide it is to divide the denominator of the map scale by 600. Suppose the map is $\frac{1}{25000} = \frac{25000}{1} = 4.16$, $\therefore$ 4 ft. apart is a suitable distance for contours on a 25-inch map. On very steep slopes, however, this must be increased. The way to do it is to set up pegs at the same altitude by means of the level and then survey them. Sometimes, however, we interpolate them as in fig. 43.* X and Y being two points whose levels are known, say X = 92.00 and Y = 120.00, place the lowest line of the diagram on the lowest level and shift it about till Y comes on the line corresponding to its level (each line being 1 foot higher, so to speak), but at the same time the line XY must be parallel to the vertical guide lines; we can then prick off our contours. The diagram is drawn on tracing-paper.

* Surveying, J. Whitclaw, 2nd ed., p. 155.

Cross-sectioning is also another use for the level. They are simply short sections at right angles to the line of main sections. They give a better idea of what the ground is like, and are necessary to calculate earthworks from.

Levels are usually taken at each place where the slope of the ground changes.

Owing to the earth's curvature we have a difference between the true and apparent level. Its value is $\frac{D^2}{2d}$, D being the distance between the two points and d = diameter of earth (7926 miles at the equator and 7900). But for practice we formulate it thus,

$$\frac{1}{2}D^2 \quad (11)$$

= the correction in feet per mile; D = distance between the two points in miles. This = 8 in. for 1 mile, for two miles $2^2 \times 8 = 32$ in., and so on. It must be added to the actual readings in very accurate work. $\frac{1}{2}$ of this is deducted for refraction. These are only considered in long lines.

Radius of a Spirit-level.

Every spirit-level has a radius so that the bubble will rise to the top. To find the radius set up the level as in fig. 44. Note the reading G, set the instrument off level and note the reading H. Measure the distance in ft. from level to staff, and note how far along the tube the bubble has shifted. Call the radius of the spirit-level R. Then S being the deviation of the level spoken of,

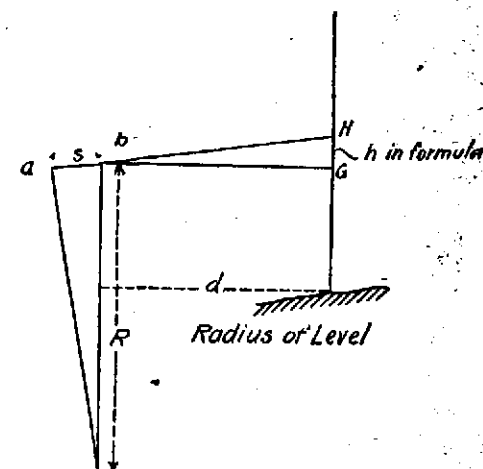


FIG. 44.

$$R = \frac{Sd}{h} \quad (12)$$

The sensibility of a level is proportional to such a radius, the level being equal to a plumb line of length = R.

Only second in importance to getting out sections is the use of the level for getting out gradients, level reaches, and pegs.

The height of such pegs is usually referred to Ordnance datum or the datum of the section. The first thing to do in any large works, after the lines have been marked out on the ground in accordance with the drawings, is to set a peg as a bench mark for the whole works, to which all levels will be referred.

We find from our section which is the best place and the most convenient height at which to set such a peg. Take, for instance, the cutting of a trench. From the section, the foreman in charge knows approximately how far down to go with his cutting. This he does by means of his two-foot rule, etc.

mark all the other levels are subsequently referred. Owing to its extreme importance, it should be set in a place well out of the way of horses, men, and tackle.

Reverting to what was previously said respecting the use of boning rods in trenches, especially those for drainage work, what are termed sight rails are used in place of the two end rods and only one rod used. They are set up in two pipes as in fig. 46. The engineer very carefully has the tops of these two rails set to the correct level according to gradient. Immediately below them are set pegs. Then a boning rod of a length = distance from top of rail to top of peg is provided. The procedure is then the same as before, simply sighting over the rails instead of two boning rods.

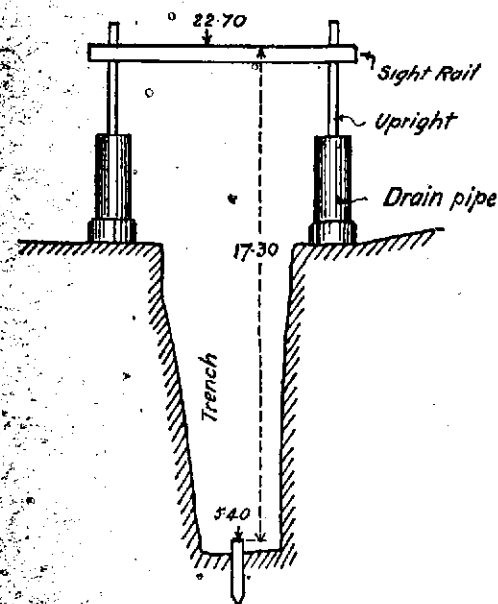


FIG. 46.—Use of Sight Rail.

Taking our previous example of a trench 420 yards long, rising 1 in 200. If we set one sight rail at 24.01 at the lower end, we know the total rise is 6.30 feet, therefore the one at the top

end must be $24.01 + 6.30 = 30.31$ feet above datum.

The boning rod used would be $17' 3\frac{1}{2}''$ long.

Driving Headings for Sewers.—Another method (fig. 46A).^{*} The line of the sewers should first be laid out on the surface of the ground, and pegs driven in to mark the intersection of the centre lines where the direction changes, and the centre of the proposed shafts. Before commencing the excavation for the shafts permanent pegs should be driven in (and covered or protected in some way) at points about 6 ft. on each side of the centre pegs along the centre line of the sewer, and along the centre line produced where the direction changes. The exact centre line is indicated by a nail driven into the top of each peg in positions found by ranging the line in with a theodolite, or if no theodolite is available, by sighting it with ranging rods. The shafts, which should not be less than 8 ft. long, can then be set out and sunk. The centre line must then be again ranged in on the surface, and

^{*} *Building World.*

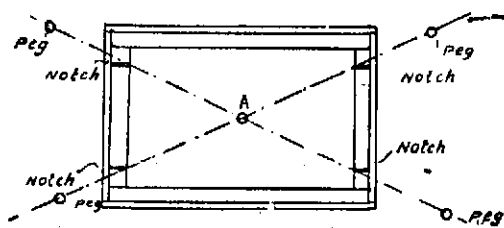


FIG. 46A.—Driving Headings.

notches cut in the top wallings of the shaft where the line cuts them, as shown in the accompanying illustration. A heavy plumb bob should be suspended from these notches, and nails driven horizontally into the bottom walling immediately below the notches. The line that is sighted through the cords of two plumb bobs suspended from these nails will then correspond with the centre line as set out on the surface, and will be the line on which to drive the heading. As the work proceeds, the centre line is ranged up the heading by using a third plumb bob, the position of which, when in line with the original two, is marked by a nail driven horizontally into the head tree. A bench mark should be made in the bottom of the shaft near one of the corners by driving a peg firmly into the ground and protecting it from damage, the level of the top of the peg above Ordnance datum being found by levelling on the surface from an Ordnance bench mark to the top walling of the shaft, and then measuring down by a steel tape on to the peg. From this peg the levels will be taken in the usual way, and sight rails fixed by nailing boards across the side trees, which should be notched level with the top of boards, so as to at once show whether any accidental movement of the boards has taken place. There should be several sight rails through the heading, so that a check is obtained on any settlement of the timbering.

CHAPTER V.

ADJUSTMENT OF SURVEYING AND LEVELLING INSTRUMENTS.

ALL instruments when purchased will probably be found a perfect adjustment, and it is the business of the maker to see that they are so. By constant use, however, they are liable to get out of adjustment, though, by careful use, the effect of wear should be trifling. Adjustments which take place in the field, such as levelling up, etc., are known as temporary adjustments, and have been described. When abroad, it is especially necessary to be able to keep instruments in perfect order and accuracy, because it is very unlikely to find instrument-makers handy. This is a further reason for always buying the best, which are, of course, less likely to go wrong.

Permanent adjustments are performed as follows: *

• **"The Adjustment of a Theodolite.**—The instructions that are given below are of a general character, for if the main principles are comprehended, adjustment is simply a matter of execution in such manner as the mechanical construction of the instrument permits.

"First, the azimuth axis (that about which the whole instrument turns) must be truly vertical. Were this not made so, it is evident that the circles described by the telescope when revolved in azimuth would be oblique instead of, as they should be, parallel to the horizon. Secondly, the line of collimation must be perpendicular to the transit axis, about which the telescope turns, because were the line of collimation oblique to the axis, it would not, on turning the telescope over, lie in the same vertical plane. Thirdly, the transit axis must be at right angles to the azimuth axis, because at any other angle arcs of altitude traced by the telescope, instead of being vertical, would be oblique to the horizon.

"Adjustment of a theodolite, then, chiefly consists in rectifying the instrument in such a way that in working it the conditions that are named above are complied with; and when an instrument is to be adjusted, it is advisable either to choose a cloudy day or at any rate a place where the direct rays of the sun will not fall upon it. The metal when exposed to the sun becomes unequally heated, and this is apt to disturb the air bubbles of the levels as well as to alter their magnitude. Before attaching the theodolite to its stand, the legs of which should be well spread out and firmly pressed into solid ground, it is a good plan to set the head by a pocket-level, a proceeding which will materially reduce the movements of the parallel plate screws in levelling. The first operation will be to set the eyepiece so that the wires are distinctly seen, that is to say, exactly to focus. This is best done by directing the telescope skyward, and then pulling out or pushing in the

* *Builder's World.*

eyepiece. Then direct the telescope to some object and focus the object glass so as to bring the image of the object to the plane of the wires and to the focus of the eyepiece.

"If this focussing is not perfectly done there will be a parallax displacement of the image when the eye is moved about the sight-hole of the eyepiece, which would impair an observation. If the image is seen to follow the movements of the eye relatively to the wires, the focus is beyond the wires; and when the contrary is the case, the focus is between the wires and the eyepiece. In the former case, therefore, the distance between the object-glass and the wires must be reduced, and in the latter case increased, by the focussing mechanism. The distance between the eyepiece and wires depends to some extent upon the power of the observer's eye, and consequently varies slightly with different individuals; and the distance between the object-glass and the wires depends on the distance of the object from the lens.

"**Correcting Azimuth Axis.**—1. In order to make the azimuth axis truly vertical, clamp the lower or graduated plate and free the upper or index plate. Bring one of the levels on the index plate parallel to two opposite levelling screws, and the other level will then be parallel to the other screws. Turn the screws so as to bring the air bubbles to the centre of their respective tubes. Turn the instrument round 180° in azimuth, and observe whether the bubbles retain the same place in their tubes; if not, correct half the error by the levelling screws, and the other half by the adjusting screws of the levels. Repeat the operations until the adjustments are perfected so that in all azimuths the bubbles retain the same positions in their tubes. Finally, test this adjustment by the large level attached to the telescope, which is usually more sensitive than the others. Set the telescope over a pair of levelling screws and centre the bubble by clamp and tangent screw to the vertical circle. Turn the instrument (index plate only, as before) through 180°. If the bubble has now moved from the central position, correct half the displacement by the tangent, and the other half by the levelling screws. Try in all positions, and make any slight adjustment that the index plate levels require by their own adjusting screws.

"2. In order to correct the line of collimation, direct the telescope to some well-defined mark; the point of a lightning conductor or the tip of a finial on some building at a distance answers very well; or at a distance of 200 ft. or 300 ft., a pinhole in a bit of paper stuck on a wall may be used. By means of the clamps and tangent screws bring the intersection of the wires accurately on the mark. Turn the telescope over either by changing pivots or changing face; the former method is alone applicable when the telescope of the instrument will not transit or revolve completely over. Either of these operations makes what was the upper side of the telescope the lower, and it is to be understood that the clamp to the index plate is to remain fixed. Bring the telescope's field of view again upon the object, and if the mark comes exactly on the wires as before the collimation is correct. If not, correct half the error by the tangent screw to the index plate, and the other half by the small screws near the eyepiece that move the wire plate, and repeat until the collimation is perfect. The operation that has been just described corrects the collimation in azimuth only.

"**Collimating Y Theodolite.**—In the case of the Y, or, as it is sometimes called, a cradle theodolite, collimating is effected in a different manner. An object is intersected by the cross wires, and the telescope held in place by

For example, first we had, say,

$$A = 19.04.$$

$$B = 18.72 \text{ and } 18.00 \text{ at second setting.}$$

$$C = 20.01.$$

Then B is 0.32 higher than A, and C is 2.01 lower than B.

Then setting up at X for A reads 21.04.

B should read 20.72, and C should read 22.73.

If, however, the line of collimation does not coincide with the axis of the telescope, we shall not attain the calculated reading. The following is a summation of the rule for making the adjustment, taken from Mr Whitelaw's *Surveying*. In adjustment the error at C should be twice that at B.

Direction of Error at C.	If Error at C is <i>more</i> than twice that at B.	If Error at C is <i>less</i> than twice that at B.
Upwards . . .	Move wires upwards	Move wires downwards.
Downwards . . .	Move wires downwards	Move wires upwards.

We then loosen the diaphragm by means of the screw and move the wires according to the rule till the error at C is twice that at B.

3. To make the level traverse, bring the telescope over 2 screws. Level up, then turn it completely round so as it points in an opposite direction, that is, shift it through 180°. If the bubble has gone off the centre (which it will not do if this adjustment is correct), correct half the deviation by the levelling screws and half by the level capstan screws.

CHAPTER VI.

THE MEASUREMENT OF AREAS AND VOLUMES, AND THE CALCULATION OF EARTHWORK QUANTITIES.

MATHEMATICAL TABLE.

USEFUL CONSTANTS.

1 inch = 25.40 millimetres.

1 Gallon = 1604 cubic foot = 10 lb. of water at 62° F.

1 Knot = 6080 feet per hour = 1 nautical mile per hour.

Weight of 1 lb. in London = 445,000 dynes.

One pound avoirdupois = 7000 grains = 453.6 grammes.

1 Cubic foot of water weighs 62.3 lb.

1 Cubic foot of air at 0° C. and 1 atmosphere weighs .0807 lb.

1 Cubic foot of hydrogen at 0° C. and 1 atmosphere weighs .00559 lb.

1 Foot-pound = 1.3562×10^7 ergs.

1 Horse-power-hour = 33000 x 60 foot-lbs.

1 Electrical unit = 1000 watt-hours.

Joule's Equivalent to suit Regnault's H is $\begin{cases} 774 \text{ ft.-lb.} = 1 \text{ Fah. unit.} \\ 1393 \text{ ft.-lb.} = 1 \text{ Cent. "} \end{cases}$

1 Horse-power = 33000 foot-pounds per minute = 746 watts.

Volts x ampères = watts.

1 Atmosphere = 14.7 lb. per square inch = 2116 lb. per square foot = 760 mm. of mercury = 10^6 dynes per sq. cm. nearly.

A column of water 2.3 feet high corresponds to a pressure of 1 lb. per square inch.

Absolute temperature, $t = \theta^\circ \text{ C.} + 273^\circ$ or $\theta^\circ \text{ F.} + 460^\circ$.

Regnault's H = 606.5 + .305 $\theta^\circ \text{ C.} = 1082 + .305 \theta^\circ \text{ F.}$

$p u^{1.016} = 479$.

$$\log_{10} p = 6.1007 - \frac{B}{t} - \frac{C}{t^2},$$

where $\log_{10} B = 3.1812$, $\log_{10} C = 5.8882$.

p is in pounds per square inch, t is absolute temperature Centigrade.

u is the volume in cubic feet per pound of steam.

$\pi = 3.1416$.

One radian = 57.30 degrees.

To convert common into Napierian logarithms, multiply by 2.3026.

The base of the Napierian logarithms is $e = 2.7183$.

The value of g at London = 32.182 feet per sec. per sec.

the clamps to both circles. The clips that hold the telescope in *Y* are released so that it can be rotated or twisted in its supports. If the intersection remains exact the telescope is correctly collimated in altitude and azimuth; if not, correct half the deviation by the wire plate adjusting screws. This method of correcting does not necessarily bring the line of collimation perpendicular to the transit axis. The setting of the telescope is the business of the instrument-maker; but as in this pattern of theodolite the telescope cannot be used inverted, a slight possible error in setting will not affect the observations perceptibly. Collimating in this case is making the intersection of the wires coincident with the axis of the cylindrical rings that rest in the *Y* supports.

"3. Setting the transit axis at right angles to the azimuth axis is, in small instruments, an adjustment that is rarely required after they have left the maker's hands; in fact, in some cases no special provision is made for effecting it. Larger instruments are provided with an axis level, which rests on the pivots of the telescope. This adjustment is known to be correct when the intersection of the cross wires will follow a truly vertical line (a suspended plumb line, for example) as the telescope is moved up and down. If a plumb line is used it should be 20 ft. or 30 ft. long, the plummet being steadied by immersion in a vessel of water. Unless the atmosphere is very still it is difficult to deprive the suspended line of all motion, and for this reason this adjustment is best done as follows. Direct the telescope to a well-defined mark at or near the summit of a building, manipulating the tangent screw of the index plate as required. Turn the telescope to sight a point near the base, which must be marked. Invert the telescope either by changing face or changing pivots, and again observe the upper mark; clamp, and then sight the lower one. If the intersection of the wires comes upon the lower mark, the adjustment is correct; but if not, correct half the error by the adjusting screw to the slipping piece on which one of the pivots rests.

"Setting the Spirit-level.—4. In setting the spirit-level attached to the telescope parallel to the optical axis, when the instrument is well levelled, and all the previously mentioned adjustments are made, take the altitude of any suitable object. Reverse the telescope, and again take the altitude as before; half the sum of the altitudes is the true altitude. Depress the telescope by this amount, and then set the bubble of the spirit-level to the centre of its tube. In taking readings for this adjustment it is necessary to pay attention to the mode of graduating the altitude circles. In some cases the graduations show zenith distances, not altitudes and depressions.

"5. In order to adjust the index of the vertical circle, set the index of the vernier to zero on the circle, and restore the level by the set screw (for which a special key is usually provided) that holds the arm to which the tangent screw is attached. In *Y* pattern theodolites this adjustment is effected by altering the vernier to read zero when the level indicates horizontality. Some transit theodolites are fitted with a level to the vernier arms to the vertical circle, which will require adjusting when the instrument is well levelled and the vernier set to zero by its adjusting screws.

"Operations 4 and 5 collimate the telescope in altitude. This adjustment might be effected by adjusting the wire plate; but this is not worth attempting, as it is a rather troublesome operation unless the horizontal wire is perceptibly out of the centre of the field of view. Adjusting the wire plate requires in all cases great care and a light hand. One screw ought not to be tightened until its opposite is slackened. The holes through which the

crews pass in the tube of the telescope are wide enough to give play, and unless attention is given to this the wire plate may get twisted round slightly, so that although the intersection of the wires may be correct, the horizontal and vertical wires may be put out of truth.

"The following is an example of adjustment No. 5 with zenith distance measurements.

Vernier II. $78^{\circ} 1' 0''$, Vernier I. $258^{\circ} 1' 0''$.

After inversion,

Vernier II. $258^{\circ} 58' 0''$, Vernier I. $101^{\circ} 58' 0''$.

The sum of these readings fall short of 360° by one minute, so that the true zenith distance was $78^{\circ} 1' 30''$. The verniers were then set to $258^{\circ} 58' 30''$, and the cross wires brought to the mark observed by the adjusting screws, and the level on the vernier arms set true. The index was then set to zero, and the level on the telescope adjusted. This, as will be noticed, is a variation from the operation as described, in which case it was assumed that a level to the vernier arms was not provided."

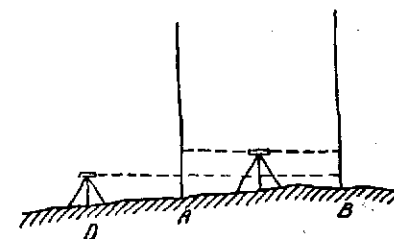


FIG. 47.

Adjustment of a Level.—1. To set the level parallel to the collimation line.

Select two points, A and B, fig. 47. Set up the level halfway between them, and find the difference of level in the usual manner.

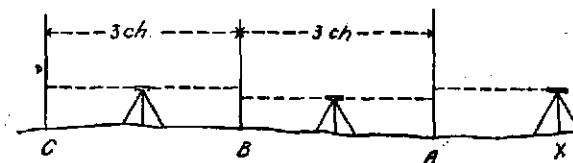


FIG. 48.—Level Adjustments.

Note it down, and shift the level to D, as near to the staff A as the figures and divisions can conveniently be seen. Read the staff at A and calculate what B should read. Assume that at the first setting the readings were A 20.06 and B 17.04. Difference of level 3.02, that is to say, B was 3.02 ft. higher than A. At the second setting A reads 14.87, then B should read 11.85. Now take the actual reading of B. It will, if the level is in adjustment, read 11.85. If not, by means of the level screws shift it off level until this reading be attained, and then bring the bubble to the centre of its run by means of the level capstan screws.

2. And to make the collimation line coincide with the axis of the telescope (refer to fig. 4b).

At A, B, and C set up 3 pegs at equal distances apart, say 3 chains. Set up between A and B, find the difference of level and note it. Set up midway between B and C, and repeat the operation. Finally set up at X, read the 3 staffs, and compare our readings with what we calculated they should be in a similar way to the previous paragraph.

We now come to the methods of making various calculations in reference to areas and volumes, and the calculation of quantities, so far as they are likely to occur in an engineer's office. It is, of course, assumed that the reader has an elementary knowledge of mathematics and geometry. To refresh his memory, however, a list is given of the most common measurements and useful constants likely to confront him in his everyday work. It only remains to say that all calculations should, where possible, be worked out in decimals.

The slide rule is also an instrument which should be learned by every engineer. Its intelligent use makes it a most valuable assistant, as it saves time and worry in rough calculations, and is a valuable check on more important ones. Its use is allowed in most examination centres, and its value has only to be known to be appreciated, because the time allowed in which to answer a stipulated number of questions is short enough without the trouble of making laborious calculations.

In taking out areas in surveys it is usual to break up the survey into rectangles and triangles. Where lines are irregular it is usual to rule

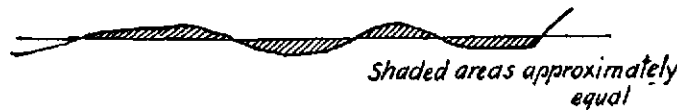


FIG. 49.—Give and Take lines.

straight lines across them so as to divide equally the irregularities. They are termed "give and take" lines, as in fig. 49.

The area of a triangle is the base $\times \frac{1}{2}$ the perpendicular height, or as in fig. 50, $\frac{AC \times BN}{2}$, or if AB and AC are known, and the angle at A, it is equal to

$$\frac{AB \times AC \times \sin A}{2} \quad (13)$$

or if the three sides are known,

$$\text{Area} = \sqrt{S(S-a)(S-b)(S-c)} \quad (14)$$

where $S = \frac{1}{2}$ the sum of the sides, and a , b , and c each of the measured sides.

This is useful in calculating areas of large triangles in the field.

Again, if we have two sides of any triangle and the included angle, and want to find the other side, then, taking the notation for triangles, which is always used in trigonometrical calculations, of the sides being known as

a , b , and c ,

and the angles opposite to these sides as

A , B , and C ,

then

$$a = \sqrt{b^2 + c^2 - 2bc \cos A} \quad (15)$$

or, having three sides and wanting the angles,

$$\cos A = \frac{b^2 + c^2 - a^2}{2bc} \quad (16)$$

The area of a square = sides squared.

" " rectangle = length $\times$ breadth.

" " parallelogram = length $\times$ perpendicular height.

" " trapezoid, which has two sides parallel = mean length of parallel sides $\times$ perpendicular height between them.

If none of the sides are parallel, draw a diagonal and make two triangles of it, as in fig. 51.

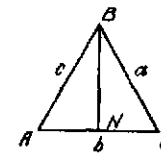


FIG. 50.

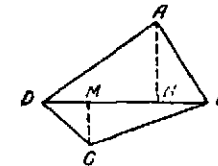


FIG. 51.

$$\text{Area} = \frac{1}{2} BD(AN + CM) \quad (17)$$

Irregular polygons are divided into triangles.

Regular polygons have an area equal half the sum of sides $\times$ perpendicular to the centre.

Now the circle has the following properties:—

If d = diameter.

c = circumference.

a = area.

s = side of a square equal in area to such a circle.

s_1 = side of a square inscribed within the said circle.

π ratio of diameter of a circle to circumference = 3.1416,

then we have the following values,

$$a = d^2 \times .7854 \quad (18)$$

$$c = 3.1416 d = \pi d \quad (19)$$

$$s_1 = .7071 d \quad (20)$$

$$s = .8862 d \quad (21)$$

We also have what is called the sector of a circle, which is that portion

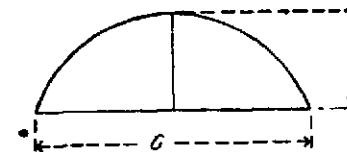


FIG. 52.

bounded by any two radii and the part of the circumference which lies between them. Then, if d = diameter of the circle = 2 radius, and N = number of degrees at centre subtended by the arc, the area

$$a = .0021816 \cdot d^2 N \quad (22)$$

A segment of a circle is that which is bounded on one side by a chord and part of the circumference (fig. 52). V is what is called the versed sine.

Then if C = length of chord, the area of the segment is

$$\frac{2}{3}CV + \frac{V^3}{2C} \quad (23)$$

This gives a close approximation only.

In a segment greater than a circle, find the area of the whole circle (Equation 18) and the area of the smaller segment, and subtract the two.

This problem occurs in finding the wetted perimeter and hydraulic mean depth of pipes and culverts.

In the case of a circular ring or annulus, such as occurs in differential pistons, if D = diameter of larger circle and d = diameter of the smaller, then the area

$$= D^2 - d^2 \times .7854 \quad (24)$$

To find the length of the arc of a circle, such as occurs in setting out curves under ground (Chapter III.), find the number of degrees at the centre

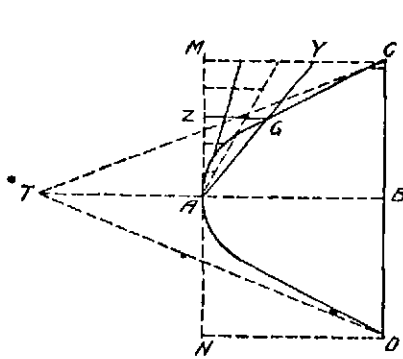


FIG. 53.

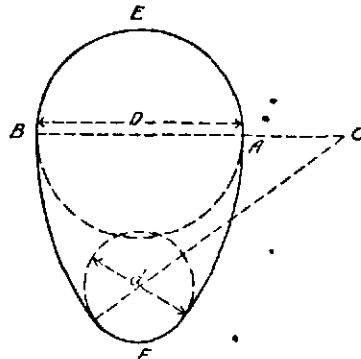


FIG. 54.

of the arc, multiply by the radius and by .017453, or if D = number of degrees and r = radius, we can formulate it thus,

$$D \times r \times .01745 \quad (25)$$

To find the versed sine of any circle for use in Equation 23, find the radius = R and then the length of half the chord = S ,

$$V = R - \sqrt{R^2 - S^2} \quad (26)$$

Fig. 53 shows what is called a parabola.

The area of the segment CAD

$$= \frac{2}{3} CMND \quad (27)$$

$$= \frac{2}{3} CD \times AB \quad (28)$$

$$= \frac{2}{3} \text{ area of triangle CTD} \quad (29)$$

$$\text{Area ACM} = \frac{1}{3} CB \times AB \quad (30)$$

Another circular property which frequently confronts the engineer is the

egg-shaped sewer. Its most usual form is shown diagrammatically in fig. 54. The properties are as follows:—

$$\begin{aligned} \text{Diameter AB} &= D \\ \text{EF} &= D + d \\ \text{CB} &= \text{EF} \end{aligned}$$

$$\text{Area} = 1.14853 D^2 \quad (31)$$

$$\text{Area of part below AB} = .755831 D^2 \quad (32)$$

$$\text{„ „ above AB} = .3927 D^2 \quad (33)$$

$$\text{The perimeter} = 3.9606 D \quad (34)$$

$$\text{Perimeter of AFB} = 2.390 D \quad (35)$$

The cubic contents of a cone or pyramid are, area of base $\times \frac{1}{3}$ perpendicular height.

The curved surface of a cone = π times the radius $\times$ slant height = $\pi r l$.

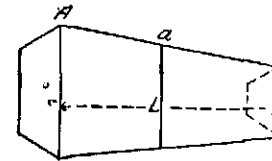


FIG. 55.

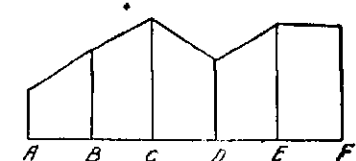


FIG. 56.

Fig. 55 shows what is called a prismoid. It frequently occurs in earthwork calculations. To find its solidity proceed as follows:—

To the area of each end add four times the middle area, multiply by the length, and divide by 6. Formulated it appears thus,

$$\frac{L(A_1 + 4a + A)}{6} \quad (36)$$

all sections being taken *parallel* to each other. This is what is called the *prismoidal formula*, and is the one generally used for the calculation of ordinary earthworks, embankments, cuttings, etc. For the calculation of large cuttings and banks we divide it into any number of equidistant parallel sections, and find the area of each, that is, A, B, C, D, E, F, fig. 56. To the end sections, A and F, add twice the sum of the sections B, C, D, E, etc. Then take intermediate sections, add four times the sum of these, multiply the whole by one-sixth of the common distance between them (viz. $\frac{1}{6}AB$, or $\frac{1}{6}EC$), and we have the solid content.

The following is Simpson's rule for the calculation of areas (see fig. 57). Divide the fig. into a number of parallel strips of equal width. The area of figure =

$$\frac{\text{First ordinate} + \text{last ordinate}}{2} \times \text{sum of all other ordinates} \times \text{width of strip,}$$

or, to formulate again,

$$\text{Area} = \left(\frac{AB + YZ}{2} + CD + EF + \dots + WX \right) \frac{l}{n} \quad (37)$$

l = length of figure, viz. AB-YZ, and n = number of strips. To describe a parabolic curve, sometimes required in examinations (*vide* fig. 53), draw the tangent NAM at the vertex A perpendicular to AB. Through C and D draw lines parallel to the axis, cutting the tangent at A in N and M. Divide AM into any number of equal parts, say 5, and divide MC likewise.

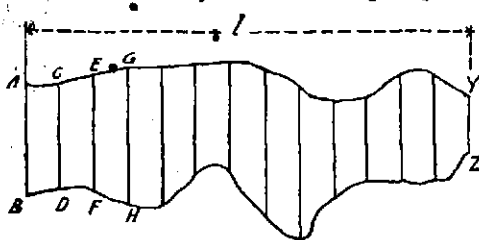


FIG. 57.

Join, say, division 3 of the latter to A, and through the 3rd division in AM draw a line parallel to the axis, cutting AY in G. Then G is a point on the parabola.

Let us take a special case of mensuration to conclude the paragraph.

"Required the contents of the cubical casting shown in

the fig. 57A.* The principle of calculating the contents of the casting shown is known by various names, such as the theorem of Pappus, theorem of Guldinus, and Tomlinson's centrobaric theorem, each of which states the case a little differently, but means the same. The essence of it is that the sectional area through one side of casting, multiplied by the circumference passed through in one revolution of the centre of gravity of this sectional area, will give the cubic contents. Making a scale drawing of the casting as in the accompanying figure, and cutting out and suspending the shaded part, the centre of gravity is found to be where marked—e.g. having a radius of 4 in. The area of the shaded part by planimeter is, after allowing for the scale, 18 sq. in. Then $\text{area} \times \text{radius} \times 2\pi = 18 \times 4 \times 2 \times 3.1416 = 452.4$ cub. in., and 452.4×263 lb. per cub. in. = 119 lb. weight of casting."

Although this is not intended to be a treatise on the use of the slide rule, of which there are many good ones published, seeing that it should be made general use of in an engineer's office, it may not be out of place to describe briefly a few of its uses. Slide rules differ somewhat in construction, but, generally speaking, they have one slide with two lines of division (which are logarithmic ones and very carefully divided), and two corresponding sets of division on the rule. In others we have two separate slides. For the purpose of the following demonstration we shall call the two fixed graduations A and D, and the two graduations on the slide B and C, as in fig. 68.

The following are the most common uses:—

1. To multiply on AB—

A	1	other number.
B	number	result.

* *Building World.*

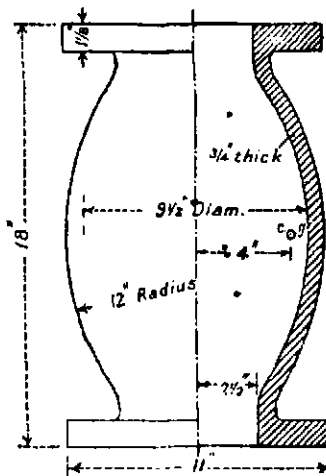


FIG. 57A.

2. To divide on A and B—

A	1	dividend.
B	quotient	divisors.

3. Length of diagonal of a square on AB.

A	99	diagonal.
B	70	sides of square.

4. To find the mean proportional (or mean square) on C and D—

C	smaller number	other number.
D	the corresponding number	mean square.

What is called the mean square is the $\sqrt{\quad}$ of the product of two numbers, viz. 9×4 ,

$$9 \times 4 = 36, \text{ and } \sqrt{36} = 6. \therefore 6 = \text{mean square of 9 and 4.}$$

5. To find the side of a square equal in area to a given circle on AB—

A	39	side of square.
B	44	diameter of circle.

6. Excavation of a trench 93 ft. long $\times$ 8 deep $\times$ 3 wide, or other size, on CD. Find the mean square of 8 and 3 by method 4 = 4.9.

C	93	83 (cubic YARDS).
D	5.196 (called a gauge point)	4.9 mean square.

(See *Slide Rule*, by Chas. Hoare, for further information.)

7. Fractions to decimals on AB—

A	required decimal	numerator.
B	1	denominator.

Example.—

A	.739	17
B	1	23

$\therefore \frac{17}{23} = .739$

8. Diagonals of a square on AB—

A	99	diagonal.
B	70	side of square.

9. Circumference of a circle on AB—

A	22	circumference.
B	7	diameter.

10. Yards $\times$ yards to acres, on AB—

A	4840	acres.
B	yards	yards.

Note.—As 69.5 is $\sqrt{4840}$, unless the yards $\times$ yards are more than 69.5×69.5 , the answer is the decimal part of an acre.

11. As applied to the lever in mechanics, see figs. 96, 97, and 98—

A	distance PF	Find W.
B	" WF	P.

12. Cubic contents of a brick wall on AB—

A. Superficial ft.	240	180	120	90
B. Cubic YARDS	5	5	5	5
No. of bricks	$\frac{1}{2}$	1	$1\frac{1}{2}$	2

13. To reduce the cubic yards so found to standard rods on AB—

A	34	cubic yards.
B	3	standard rods ($1\frac{1}{2}$).

14. Weight of cast-iron pipes—

A	constant	weight per foot run.
B	"	mean circumference in ins. (vide '3).

The constants are as follows:—

Thickness	$\frac{1}{2}$ in.	$\frac{3}{4}$ in.	$\frac{7}{8}$ in.	1 in.
A.	16	22	28	33
B.	10	12	12	12

Example.—A 9-in. pipe $\frac{3}{4}$ in. thick.

A	28	70 lbs. per foot run.
B	12	30 ins. mean circumference.

15. Area of a circle on CD—

C	785	line of areas.
D	1	line of diameters.

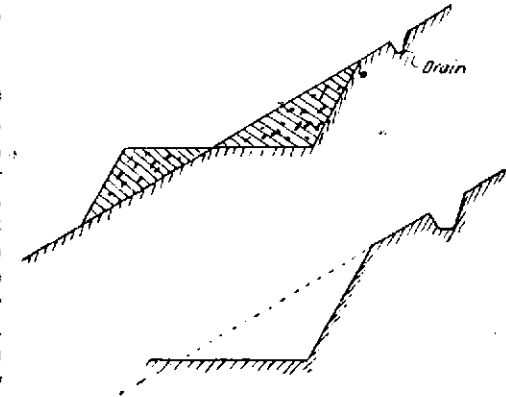
Earthworks.—All earthworks have a certain natural adhesion which allows of temporary vertical cuttings. The permanent stability is due to friction alone. (The usual slopes in engineering practice are $1\frac{1}{2}$ to 1 and 2 to 1. The corresponding angles of slope are, $33\frac{1}{2}^\circ$ and $26\frac{1}{2}^\circ$.)

The following is a list of the natural angles of slope of various materials, that is, the natural slope they would assume when loosely tipped and left to themselves. This angle is also known as the angle of rest, or angle of repose.

Nature of Soil.	Angle of Slope.	Weight per Ft. Cube.
Gravel	40°	112 lbs.
Dry sand	38°	92 "
Sand	22°	116 "
Vegetable earth	28°	83 "
Compact earth	50°	100 "
Shingle	39°	141 "
Rubble	45°	153 "
Drained clay	45°	112 "
Wet clay	16°	124 "

The best materials for banks are rock shivers, shingle gravel, and clean sand. Dry clay is also good, but on no account should wet clay be employed; it subsides and slips. The worst stuff we are likely to meet with is clay and sand in a mixture.

In rock we may have vertical sides in a cutting, especially in igneous rocks, such as granite; but sedimentary rocks, such as shale, require a little forethought before cutting. In all such cases judgment and experience on the part of the engineer are the best guides. Ground which has a sloping cross section is called *sidelong ground*, that is, ground such as appears in fig. 58. Three forms of cutting will now be explained, together with the means of getting their cross-sectional areas. This is, of course, the most important part of the calculation by the prismoidal formula (Equation 36), because the only other measurement is the length, which is simple enough.



Figs. 58 and 59.

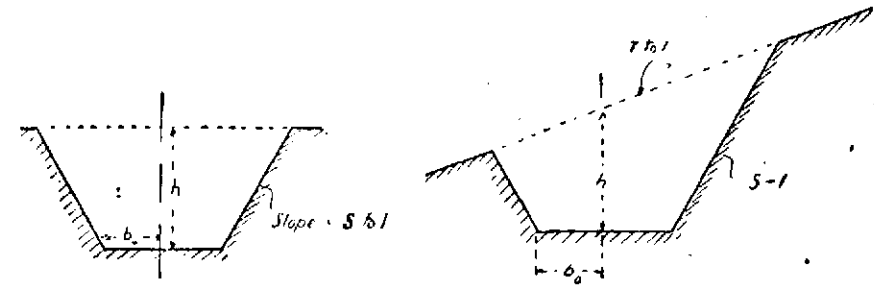


FIG. 60.

FIG. 61.

The cuttings illustrated become bank when turned upside down, and are of course calculated in the same manner.

The numeration employed, with following formula, which is deduced from Rankine's *Civil Engineering*, is as follows:—

Actual slope of ground	r to 1
Slope of sides	s to 1
$\frac{1}{2}$ breadth of base	b_0
Centre depth	h

Then in figure 60 the cross-sectional area is equal to

$$(2b_0h) + (sh^2) \quad (38)$$

In fig. 61,

$$\frac{sb_0^2 + 2r^2b_0h + r^2sh^2}{r^2 - s^2} \quad (39)$$

In fig. 62 we have two quantities, viz.

$$\text{Bank} = \frac{(l_0 + 2h)^2}{2(r-s)} \quad (40)$$

$$\text{Cutting} = \frac{(l_0 - rh)^2}{2(r-s)} \quad (40a)$$

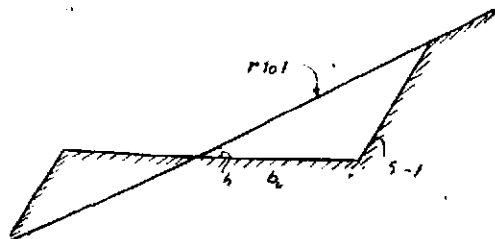


FIG. 62.

Where, however, the ground is level, we have another method, as in fig. 63. The slopes are 2 to 1 and 3 to 1.

$$\text{Measure } h \text{ and } H = \frac{h}{3+2} + h \quad (41)$$

$$\text{and the sectional area} = \frac{H \times W}{2} \quad (41a)$$

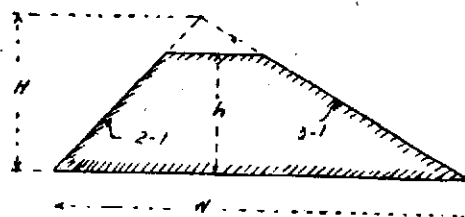


FIG. 63.

We will take an actual example, referring to fig. 61. Say

$$\left. \begin{array}{l} s = 2 - 1 \\ r = 5 - 1 \\ b_0 = 10 \\ h = 10 \end{array} \right\} \text{then}$$

$$\frac{(2 \times 10^2) + (2 \times 5^2 \times 10 \times 10) + (5^2 \times 2 \times 10^2)}{5^2 - 2^2}$$

$$= \frac{200 + 5000 + 5000}{25 - 4}$$

$$= \frac{10200}{21} = 485.7 \text{ square ft.}$$

In all engineering work of importance borings are made to ascertain the nature of the subsoils. The tool used is the auger, but for hard rocks special tools are used.

The holes are usually 4 in. in diameter. The information obtained by all such borings must only be taken as very approximate, for the following reasons:—

1. The tool generally breaks up the specimens.
2. In hard strata, water is required to keep the tool cool. This, more than not, reduces the specimens to mud.
3. The small hole often gives doubtful results by showing rock which may be only an isolated boulder. In fact, trial pits are the only satisfactory means of ascertaining the substrata, and, owing to their expense, they are not usually made except in large works. It is important, when such borings are made, only to put on the section those strata which we actually know to exist. It is injudicious, to say the least, to imagine something as existing, and to put on your drawing with the object of making it "complete."

Figs. 59 and 64 show what are termed catchwater drains. They are usually 3 ft. to 4 ft. wide and 2 ft. to 3 ft. deep. Their object is to intercept surface water and springs, and to keep the workings dry.

Cuttings are usually started with a small vertical cut called a "gullet." This is made wide enough to lay the contractor's rails in. They are always

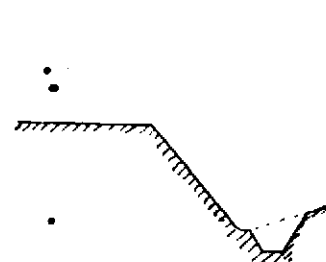


FIG. 64.

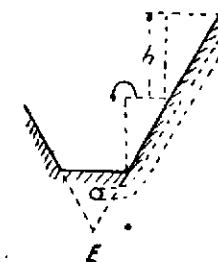


FIG. 65.

made with a longitudinal gradient. This allows of the spoil being removed by gravity. That is, a train of full waggons pulls up a train of empty ones. If such is not practicable, we have to lay a special wire-rope way running round a sheave at the top. This sheave is then worked by steam or other motive power. Such an arrangement is of course a continual expense, and a danger to workmen. Horses, of course, are often employed.

These vertical sides must be trimmed off as soon as possible, and where deep they must be done in lifts, as in fig. 65.

The depth "h" should not be more than 7 to 8 ft. The earth is tipped direct into the waggons, as shown by the arrow $\nearrow$; when much water occurs, the drains are laid, as in fig. 65. The bottom of it must, however, be in no case below E, unless there is a retaining-wall. The various labours on earthworks are—

"Getting," "filling," "wheeling" (in barrows), "leading" (in waggons), and finally tipping. Wheeling is done on planks. These must be set with a gradient. Its limit is 1 in 12. The three divisions into which we may divide the various kinds of ground met with are—

1. Loose earth, sand, soft clay, mud. This can be shifted by means of a shovel.
2. Stiff earth, hard clay, hard gravelly soil, and chalk. These necessitate the use of a pick.

3. Rocks. Blasting required here usually. Then in class No. 1 a man shifts about 22 cub. yards in 1 day of 10 hours.* In class No. 2 this comes 18.

To allow a man to work comfortably, all trenches should be at least 6 ins. wide. As regards the implements used for removing soil, etc., these are the following:—

	Approximate capacity in cub. yards.
Light wheelbarrow	$\frac{1}{8}$
Ordinary "	$\frac{1}{4}$
Heavy "	$\frac{1}{2}$
Dobbin cart	$\frac{3}{4}$
Horse cart (usually $6 \times 3\frac{1}{2} \times 2\frac{1}{4}$)	1 $\frac{1}{4}$
Earth waggon (filled only)	2
" " (heaped)	2 $\frac{1}{2}$

Barrows are used for removal up to 100 yards.

Dobbin carts up to about 350 "

Horse cart anything up to $\frac{1}{4}$ mile.

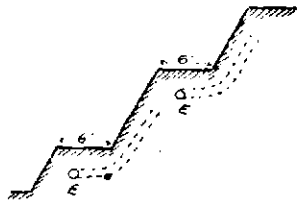


FIG. 66.

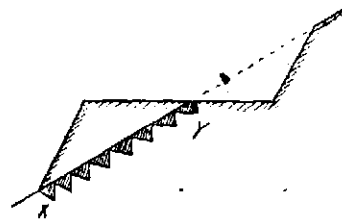


FIG. 67.

It is then advisable to lay down rails and use waggons and small locomotives.

The rate of excavation of ordinary earth would be about 20 cub. yards per day, or 16 of stiff earth (per man).

"Earth of one man" is the term used to denote that what one man is excavating is keeping a line of wheelers at work, and so on according to how many men are required to keep a line of wheelers at work. When the sides of a vertical trench are let in it is called "falling."

An earth waggon can be taken in work equal to 60 barrows having a capacity of 50 and a speed $\frac{1}{5}$ as great again.

Fig. 66 shows what are termed "benches." They are used in deep cuttings. They should, like the trench, have a longitudinal gradient, and have drains as shown, and not be more than 6 feet wide. In wet ground, dry rubble retaining-walls are an advantage. Such work is illustrated in fig. 66.

This was a very treacherous cutting, and occurred on the old London and Birmingham Railway at Blisworth.

Here we have a stratum of limestone 25 ft. thick, below it was wet clay. The result was, the heavy limestone tried to force out the wet clay; by means of rubble walls and puddle-clay drains effective drainage was carried out. There were buttresses, which were continued to the invert. This latter process is called "undersetting."

* Slagg, *Water Engineering*.

here usually. Then in class No. 1 a man engineer who is held responsible in 1 day of 10 hours. In class No. 2 this localities, and it is impossible.

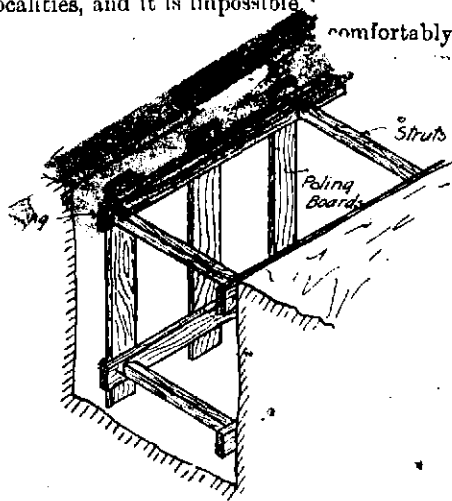


FIG. 70. — Timbering Trenches.

comfortably, all trenches should be at least 4 ft. wide, and used for removing soil, etc., these

opportunity of operation. He will be in large towns where streets are "up." I take rough sketches of what he sees and draw them in his spare time. This practice is an invaluable one, and the notes come in useful at times. The elements of the work

Struts, poling boards, and walings. Fig. 70 shows a simple case in modern practice. Here the poling boards are vertical, and at a certain distance apart to give clearances.

Fig. 71 shows the boards quite close for use in running sand. They may have horizontal poling boards and vertical walings.

Deep trenches have to be run in stages, because the limit of which a man can conveniently shift earth out of a trench is about 7 ft. when the trench is deep. Then on this stage, which is supported on the struts, another man stands and shifts out the earth from the man below, or to the man on the stage above. These struts are of course always made slightly longer than required, so as to form wedges and keep the walings quite tight. Where the soil is particularly treacherous we proceed downward by means of what are termed runners. They are close poling boards, fig. 72, and are driven down by a mallet as the excavation proceeds behind them. By this process none of the soil is exposed. When these are driven in, the soil is driven inside them; this of course tends to reduce the width, in which case a suitable allowance is always made. When a culvert is laid, the filling is carefully proceeded with and

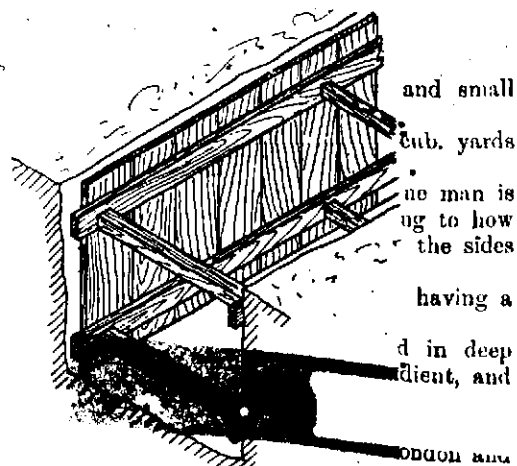


FIG. 71. — Close Poling Boards, wet clay.

CHAPTER VII.

MECHANICS OF ENGINEERING.

THE branch of mechanics which concerns the civil engineer is what is generally known as "applied" mechanics; that is to say, we assume certain facts to exist for the purpose of our demonstration and formulæ without the trouble of actually proving their existence, such subjects being fully dealt with in theoretical mechanics. It has an extremely wide scope, and is the basis of every kind of structural design that confronts an engineer. Without a thoroughly sound knowledge of the subject an engineering pupil can never hope to get on. There is no use in disguising the fact that the subject in its early stage is extremely dry and uninteresting, and this is all the more to be lamented because those who do make a start to master the all-important subject get disheartened. The one thing I would like to urge on all young engineers is to make up their minds once for all that they will master the subject in its early stages and stick to the work. It is one of the subjects more especially that require for the first year at least a competent class teacher. Most subjects can, by the ordinary intelligent person, be studied up from books, but the early stage of mechanics cannot. A year's course at a technical institute amply repays the time given to it. It widens the pupil's views; he gets interested, and is prepared to dive further into the subject from his own books in his spare time, wherein he should strenuously avoid a compilation of complex formulæ, but try and see the why and wherefore of such formulæ, because although he may safely remember them for a certain time, he will be sure to forget them sooner or later, whereas if he understands the principles, he can then construct his own equations according to the principles involved.

For instance, if he knows that the strength of a beam depends on—

1. The breadth = b ,
2. The depth squared = d^2 ,
3. Inversely as the span = L ,

and can remember a coefficient = c ,

he readily makes the breaking load on any beam at the centre

$$= \frac{cbd^2}{L}$$

which is the formula for such a case.

In a book like this it is impossible to give anything more than a summary of the branch of the science which concerns us here. Those who wish to go further afield should read any of the standard works on the subject.

Notably—

1. *Applied Mechanics*, Jamieson.
2. " " Rankine.
3. " " Perry.
4. *Design of Structures*, Anglin.
5. *Strength of Materials*, Mosley.

The two branches here which concern us principally are—

1. Statics, the science of forces at rest.
2. Hydraulics, dealing with the forces exerted by fluids.

The other branch, Kinetics, deals with moving forces, such as motive power, gearing, screws, etc., which principally, though by no means exclusively, concerns the purely mechanical engineer, or designer of prime movers and machines.

1. *Matter*.—We cannot state exactly what matter is. It always, however, occupies space and when occupying a definite space is called a body.

2. *Mass* is the quantity of matter any body contains. The standard quantity of mass is called the pound.

3. The *Density* of a material is the quantity of matter in it compared with the amount of matter in the similar volume of some other substance.

4. *Specific Gravity* is the weight of a given volume of a substance when compared with the weight of an equal volume of water, which is taken as the unit.

The following materials possess the specific gravities shown:—

Material.	Weight of One Cub. Ft. in Lbs.	Specific Gravity.	Weight of Sheet 1 in. thick, 1 sq. ft. in area.
Fresh water	62.5	1.0	...
Sea water	64.0	1.024	...
Wrought iron	480	7.7	40
Cast iron	450	7.2	37.5
Steel	490	7.8	41
Copper	550	8.8	46
Brass	525	8.6	44
Gun-metal	540	8.7	45
Aluminium	165	2.6	14
Zinc	450	7.2	37.5
Tin	465	7.4	39
Lead	710	11.4	59

This table will be useful in taking out quantities and weights of metallic structures.

5. *Force*.—A force is that which changes or tends to change the direction or speed of motion of a portion of matter.

Graphic Representation of Forces.

Every force must have—

1. *Direction*.
2. *Magnitude* (tons, lbs., etc.).
3. *Sense* (push or pull).

We can, if we know these, represent a force on paper by a straight line.

engineer will like an equality, bearing in mind that all banks settle. It is localities, allow banks 10 per cent. extra, any surplus being disposed of by leveling the slopes. In all cases we should, however, avoid having too much of earth on spoil banks. Where earthwork in any quantities have to be taken out, however, the

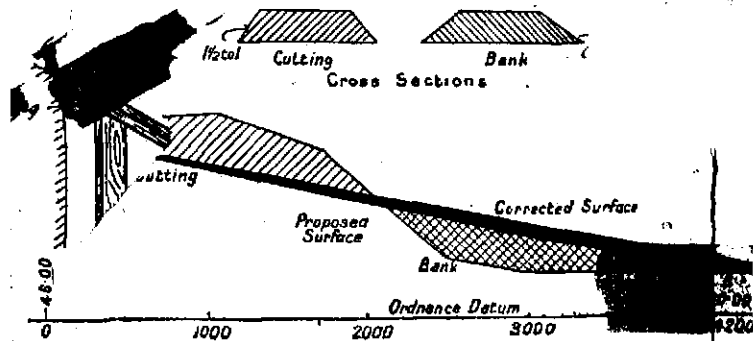
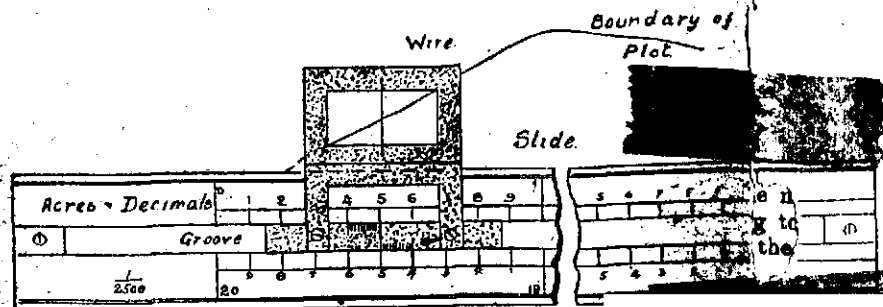


FIG. 75.

use of the prismoidal Formula is a very laborious process, to avoid which earthworks tables have been prepared, those of Mr John MacNeill being the most popular. They are a series of tabular numbers corresponding to bases of cutting of a certain width, with slope of various degrees of slant. With banks we have two tabular numbers.



tracing paper (or better, cloth) divided by fine lines into square chains on the particular scale for which the computer is intended is absolutely necessary. It should be of good size. The divisions on the scale are in chains. It can now be easily understood that, if we can find out how many chains long any particular strip is, we can calculate the acreage, provided it is divided into strips of 1 chain width, for if a parcel was 38 chains long and 1 chain wide we would know that its area was $\frac{38}{10} = 3.8$ acres. It is this division into strips of a chain wide that the scale performs, and therefore shows at once the acres and decimals. Of course skill on the part of the operator in equalising the boundaries by give-and-take lines is indispensable. That is the purpose of the fine wire (often a mark on a piece of glass), to enable the operator to gauge the amount of give and take. To use the scale, place on the plan the squared paper. Pin or weight it down. Put the scale to zero. Put it parallel to the lines on the paper (the top part). Run it along the parcel contained between these two lines; then, without in any way disturbing the slide, shift it down to the next line. Place the wire in the correct position, for marking off the give and take, and continue the process till we read 10 acres. We then have to set to zero again, and start reading where we left off. These sliding calculators save time when many areas have to be taken off a map of the same scale, but they usually only suit a particular scale (although they are made to do duty for more than one), therefore their scope is not very wide.

This gives us the stress on the pulley at A. If we draw DE parallel to the stay cutting the jib in E, DE represents the stress in that stay, and the length AE the stress on the jib.

Now if we draw from E a vertical line EX, and from A a horizontal one AX, AX gives us the horizontal pull and EX the vertical pull on the crane post.

Moments.—The moment of a force is the tendency of that force to turn the body on which it acts about a given axis.

Its value is a complex quantity that is not written lb. or ft. only, but pound-feet, ton-feet, etc., and is equal to the force multiplied by the distance from the axis of rotation to the line of action of the force measured on a line always PERPENDICULAR to that direction of action. Thus in fig. 82 we assume a rod AB hinged at A, a pulley at D, a weight at W, and a cord passing over the pulley. The effect is to turn the rod AB in a clockwise direction C.

A is the axis, CD the force, and C-A the perpendicular distance to the

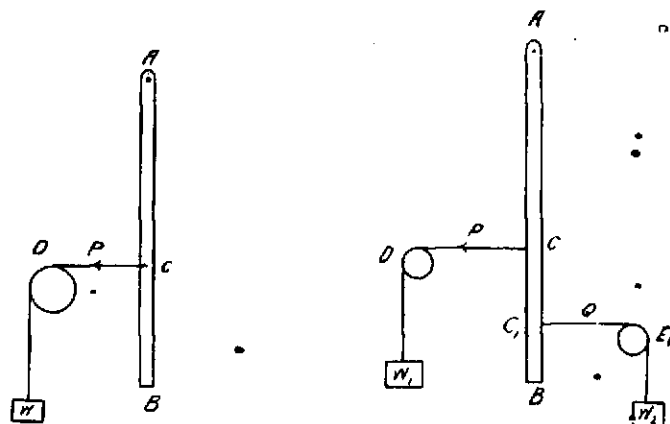


FIG. 82.

FIG. 83.

line of action of the force. Then if AC = 1 ft. and W = 10 lbs., the moment = $10 \times 1 = 10$ pound-feet. If AC was doubled this moment would be doubled. The same would occur if we doubled the weight. Thus in fig. 83

AC = 5 ft., AC₁ = 7 ft., W₁ = 7 lbs., and W₂ = 5 lbs.

$$\therefore 5 \times 7 = 35 \text{ lb.-ft.}$$

$$\therefore 7 \times 5 = 35 \text{ lb.-ft. also.}$$

$\therefore$ the moments being equal, no rotation will take place—an example of equal moments with unequal forces.

But take fig. 84, the distances AC and AC₁ no longer hold good. They are not perpendicular to the lines of force DC and EC₁, in which case we draw AM perpendicular to DC and AN perpendicular to EC₁. The moments now = W₁ × AM and W₂ × AN, the former being clockwise, the latter anticlockwise. Perhaps the most important applications of moments which confront the civil engineer are those in reference to loaded beams and walls. In fig. 85 we have a beam supported at the ends (not fixed), and loaded as shown. We want to show how the principle of moments is applied to

finding the reaction on the supports. That is, how much of the total load (3.75 tons) they bear. First find the reaction on the support P. To do so we take the moments about the opposite support Q as follows:—

$$\frac{1}{2} \times 17 = 8.5 \text{ ton-ft.}$$

$$\frac{1}{4} \times 13 = 3.25 \text{ "}$$

$$1 \times 7 = 7.0 \text{ "}$$

$$2 \times 4 = 8.0 \text{ "}$$

$$26.75 \text{ total (ton-ft.).}$$

The span = 20 ft. Now divide the moments found by the span

$$= \frac{26.75}{20} = 1.3375 \text{ tons.}$$

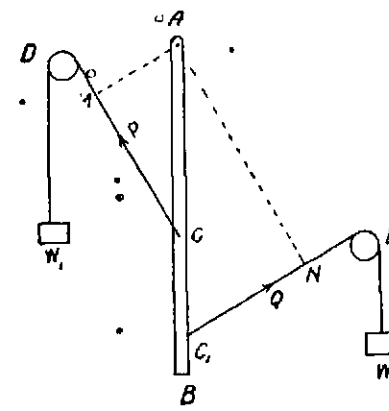


FIG. 84.

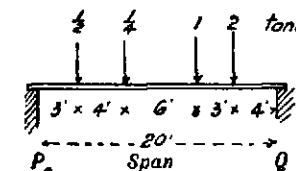


FIG. 85.

$\therefore$ reaction on P = 1.3375 tons; and as total load = 3.75 tons, load (or reaction) on Q must be equal to

$$3.75 - 1.3375 = 2.4125 \text{ tons;}$$

but to complete the example, we will work it out in full.

Taking moment about P, we have

$$\frac{1}{2} \times 3 = 1.5 \text{ ton-ft.}$$

$$\frac{1}{4} \times 7 = 1.75 \text{ "}$$

$$1 \times 13 = 13.0 \text{ "}$$

$$2 \times 16 = 32.0 \text{ "}$$

$$48.25 \div 20 = 2.4125 \text{ as above.}$$

Again, take a wall AB as on fig. 86, having water on one side. As we shall point out later, the water exerts certain pressure of say 200 lbs. per square foot.

Again, say the wall was 2 ft. thick and weighed 700 lbs. per foot-run. This acts through the centre of the wall at a distance of $\frac{1}{2} \times 2$ or 1 foot. $\therefore$ It causes a clockwise moment of $700 \times 1 = 700$ lb.-ft.

Again, the water causes an anticlockwise moment of $200 \times 3 = 600$ lb.-ft.

The length of the line represents the magnitude of the force,
The direction of the line represents the direction of the force,
And an arrow-head on the line its sense.

If pointing towards the point of application, it is a push, and if away from it, a pull; thus in fig. 76 the straight line AB represents a pull from the point A of a magnitude equal to the scale decided upon. In graphic statics these scales may of course vary according to circumstances, such as 100 lbs. = 1

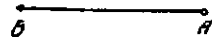


FIG. 76.

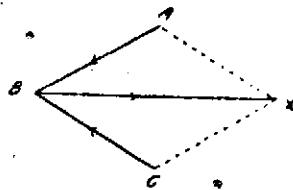


FIG. 77.

inch; 1 ton = 1 inch; 5 lbs. = 1 inch, etc.; simply a matter of a little forethought.

If two forces acting in the same line have equal values and opposite directions they balance each other, such as a tie-bar under pull or strut under compression.

If we have two forces, however, intersecting at a point, fig. 77, AB and BC being the two forces intersecting at B, we can find a single force which applied alone would have the same effect as the two forces together. This

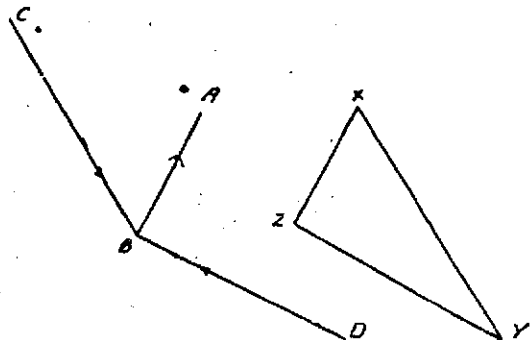


FIG. 78.

single force is called the *resultant*. It is found by means of the *parallelogram* of forces as follows.

Draw to scale and in the correct direction the two forces AB and BC (fig. 77). Draw AX parallel to BC, and CX parallel to BA.

Draw XB the diagonal of the parallelogram, then BX measured off to scale and in the direction of BX gives also the single force equal to AB and BC. AB and BC are called the *components*. This parallelogram of forces has many

hundreds of applications in engineering problems. Take the case of three forces acting at a point. One of them must be equal and opposite to the resultant of the other two. Say we have three forces as in fig. 78; we want to find out if they are in equilibrium; plot them to

scale and direction. To do this we must draw them in their proper order according to sense. That is, YX is drawn downwards to the right in the same direction that CB acts, while ZX is drawn upwards to the right for the same reason. Then these two lines being plotted to scale, if YZ drawn equal

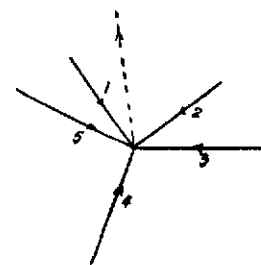


FIG. 79.

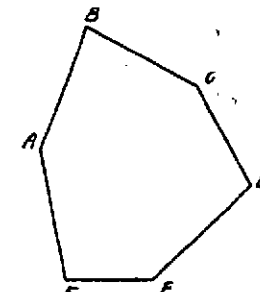


FIG. 80.

in magnitude to BD in the correct direction and sense encloses a triangle XYZ, the three forces CB, BA, BD are in *equilibrium*. Now, in the same way as before, we can find the resultant of any number of forces. Find the resultant of any two; then another two the same way; then find the resultant of the resultants, and so on.

What we called the *Polygon* of forces is merely an extension of the triangle of forces. Thus we can find if any number of forces acting at a point are in equilibrium. Refer to fig. 79. Here we have 5 forces acting at a common point.

By means of a parallel ruler plot them as in fig. 80, taking them in rotation, starting say at No. 4 = AB, 5 = BC, etc. If at the end of our plotting we have enclosed a polygon, then these forces are in equilibrium. But in the case in question we have not done so. But by joining AF (fig. 80) we find the magnitude

and direction of a force applied at A which will balance them and cause equilibrium as shown by dotted lines. "As an application of the foregoing reasoning, let us take an ordinary jib crane, fig. 81. Here we have jib, stay, post, and chain, and a load of 5 tons applied, and we want to find what effect the 5 tons will have on the various parts of the crane.

To any convenient scale draw AB parallel to the suspended chain and equal to 5 tons.

Draw AC parallel to the back chain and equal 5 tons again. Draw CD and BD, and so make a complete parallelogram. We then have a line AD

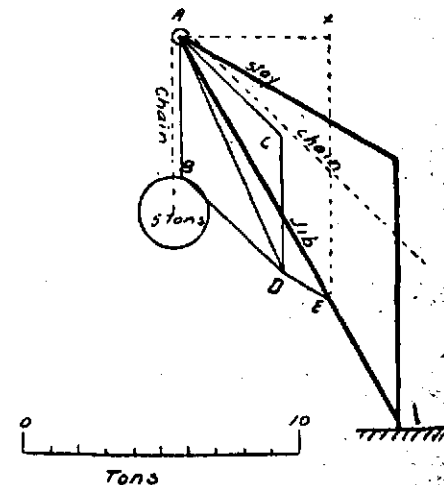


FIG. 81.

the moment due to the wall is greater than that due to the water. $\therefore$ the wall will be stable and withstand the pressure. This is, of course, only a simple case.

We shall devote a chapter to the stability of walls, but the whole of our future reasoning will depend on the foregoing principle.

Now two forces of equal magnitude applied to the same body in parallel and opposite directions constitute a *couple*. Thus, fig. 87, P and Q are a couple and CD is the *arm* of the couple, and $P \times CD = Q \times CD =$ the moment of the couple. A couple tends to cause rotation of the body upon which it acts, for it has no resultant.

Centre of Parallel Force.—In fig. 88, say, we have two forces, AX and CZ and their resultant = BY passing through Y. AX and CZ are parallel forces.

Now change their direction as shown by dotted lines, but keep them parallel, their resultant will also change its direction till it is parallel

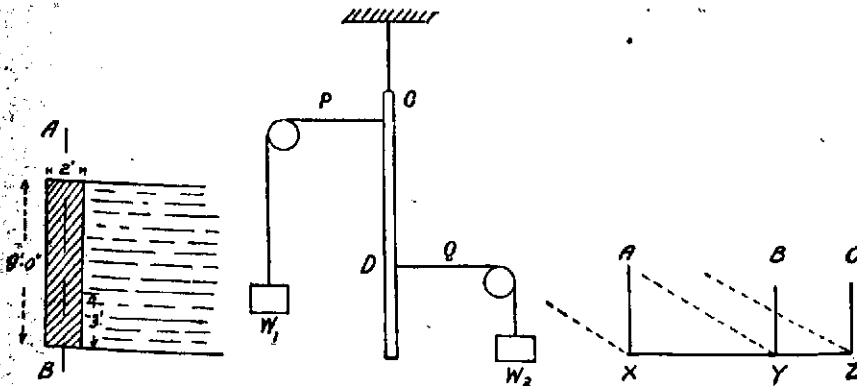


FIG. 86.

FIG. 87.

FIG. 88.

with them. But it *always* acts through the same point Y. This point is what we call the *centre of parallel forces*, AX and CZ.

Everyone knows the earth exerts a *pull* on everything upon it, and any body allowed to do so *falls* by the action of gravity. Now on a falling body gravity sets up an innumerable number of parallel forces; these parallel forces of course have a centre, called the *centre of gravity*. This centre of gravity occurs in various positions in a body according to its shape and position. Thus in regular figures it has a known position as follows:—

- In a uniform rod at the middle of its length.
- In a square at the intersection of its diagonals.
- In a rectangle and parallelogram intersection of its diagonals.
- In a circle at its geometrical centre.
- In a triangle $\frac{1}{3}$ the way up from centre of base to apex.
- Pyramid $\frac{1}{4}$ " " "
- Cone $\frac{1}{4}$ " " "

When we are considering the equilibrium of a body we regard its whole weight as concentrated at its centre of gravity.

In an irregular body the centre of gravity has to be found by experiment. Its solution in most cases wants higher mathematical knowledge.

Stress, Strain, and Elasticity.—If we hang up a bar and apply 10 lbs. to

the end of it, we have obviously a load of 10 lbs. on the bar. If that bar is 1 square inch in section area, then the *Intensity of Stress* of the bar is 10 lbs. per square inch.

If it is made of a soft material such as rubber, which will stretch or elongate very much, and if the bar is 5 ft. long = 60" and stretches 1" due to the load, we have a tensile *Strain* = $\frac{1}{60}$, or .01666. Strain is not measured in units, but is simply a ratio of the elongation (due to load) to the original length.

As no matter has been added to the bar anywhere, and as its length has increased $\frac{1}{60}$, it must be smaller in size somewhere, which is to say, there was a *Reduction in area*, a phenomenon which takes place in all bars of metal under test in a greater or lesser degree. In the same way, by supporting the bar below and loading it on top, we should have stress (compression) and a strain, but this time an *Increase* in area if the intensity of stress was great enough to cause it.

rubber will usually return to practically its load is taken off. Steel and iron and other differently.

on a bar as to cause it to elongate or shorten off it remains permanently deformed, the metal in other words, we have exceeded the *Elastic limit* or rectified by a process called *Annealing*. This is done for some hours and allowing it to cool very slowly applied to chains, which, being subject to very sudden jars, become crystalline by the elastic limit being exceeded.

Elasticity is the property all matter contains in a greater or lesser degree of returning to its original shape after being stressed under a load. The elastic limit is the point where the bar is overstressed, or where *permanent set* occurs. It is where *Hooke's law* breaks down. Again we have what is called *Young's Modulus of Elasticity*. It is usually called simply the *Modulus of Elasticity*, and in formulae appears as E. It is the load which if it could be practically applied would stretch a bar to twice its original length or compress it to half that length. If we know the stress (lbs. per sq. in.) and the strain, then—

$$E = \frac{\text{stress}}{\text{strain}} \quad (42)$$

Its value in common material is as follows:—

	E. lbs. per sq. in.	Greatest Load without Permanent Set (lbs.).
Wrought iron	29,000,000	17,000
Steel (mild)	30,000,000	81,000
Cast iron	15,000,000	5,000
Copper (rolled)	12,000,000	8,200
Brass (rolled)	11,000,000	7,100
Gun-metal	9,000,000	10,000
Phosphor bronze	14,000,000	...
Aluminium bronze	17,000,000	21,000

Col. 2* is known as the ultimate strength; the elastic limit may be anything from 50–70% of this.

All materials have this property, that a load which will not break a bar if gradually applied will do so if continually applied and taken off. The greater the number of reversals of stress, the smaller will be the load which will cause rupture.

Of course, in engineering structures we do not load the members to anything like the breaking load, but have what we term *factors of safety*. For instance, in building a bridge of mild steel of 30 tons ultimate tensile strength, the working stress would be about 5 tons per square inch. $\frac{30}{5} = 6$ = a factor of safety of 6.

They vary, but are generally taken as follows:—

	Live Load.	Dead Load (only).
Metals	6	3
Timber	8-10	4-5
Masonry	8	4

Material.	Ultimate Statical Strength.	Similar Stresses.		One Stress = Zero.		Opposite Stresses.	
		Least.	Greatest.	Zero.	Greatest.	Least.	Greatest.
1. Wrought iron (Ord.)	22.8	12.0	20.5	0	15.3	- 8.6	+ 8.6
2. Axle steel (Krupp)	52.0	17.5	37.8	0	26.5	- 13.1	+ 13.1
3. Spring steel (untempered)	57.5	12.5	34.8	0	25.6	- 12.4	+ 12.4
4. Iron plate	22.8	11.4	19.2	0	13.1	- 7.2	+ 7.2
5. Bar iron (1)	26.6	13.3	22.0	0	14.4	- 7.9	+ 7.9
6. Bar iron (2)	26.4	13.2	21.9	0	15.8	- 8.7	+ 8.7
7. Mild steel plate (Bessemer)	28.6	14.3	23.8	0	15.7	- 8.8	+ 8.6
8. Steel axle	40.0	20.0	32.1	0	19.7	- 10.5	+ 10.5
9. Steel rails	39.0	19.5	30.9	0	18.4	- 9.7	+ 9.7
10. Mild steel boiler plate	26.8	13.3	22.6	0	15.8	- 8.7	+ 8.7

Material.	Values of n .
1	1.33
2	1.83
3	2.14
4	1.6
5	1.67
6	1.53
7	1.68
8	1.91
9	2.00
10	1.53

Strength of Materials.—To understand the sequel, the following symbols and their values must be noted:—

f min. = minimum stress in the range of stress considered (this will be

proportions are say, 3-4 live and dead load, viz. 30 tons dead, with a factor of safety of 5 and 40 tons live, with a factor of safety of 4. Then

$$\frac{5 \times 3 + 4 \times 4}{5 + 4} = \frac{15 + 16}{9} = 3\frac{1}{9}$$

by a combined factor of safety of 3.5 for that particular structure.

Now when we load a bar up to its proof stress, mechanical work is done. say the load = 100 tons = P . The bar is 1 sq. in. in sectional area. The its length 10 in. = l . we have a property in that bar which enables it coming back to its original form on removal of the load. This is termed the resilience of the bar.

$$\text{Its value} = \frac{1}{2} Pl \quad (46)$$

per sq. in. = work done.

Given f = stress in lbs. per sq. in.

E = lbs. per sq. in. as usual (vide table)

have a quantity

$$= \frac{f^2}{E} \quad (47)$$

this is called the *Coefficient of Resilience*, the actual resilience being

$$\frac{f^2}{E} \times \frac{1}{2} \text{ volume of bar} \quad (48)$$

We also stated before that the effect of a suddenly applied load was greater than the same load gradually applied. It is in fact twice so. But the resilience is 4 times, owing to the fact of the load being doubled (2), and the elongation doubled (2) = $2 \times 2 = 4$.

As an application of the use of our values f and E , take the example of crank rings. On turbine rotors and similar work, where centrifugal force tends to set up much tension in a cast-iron rotor, C.I. being weak in tension (tons per sq. in. being the safe load), we have to shrink on to *strain discs* to counteract the effects. Say, now, grooves were turned in the diameter of D inches, and rings turned to an internal diameter d inches, the strain on the rings is

$$\frac{D - d}{d} \quad (49)$$

If we want to design the rings to grip when cold (they are applied hot) say 5 tons per sq. in. = f . Then before heating up, the correct diameter at ring must be

$$d = D \left(\frac{E}{E + f} \right) \quad (50)$$

positive if f min. is of the same kind as f max.; a negative of the st of the opposite kind).

Δ = range of stress = f max. $\pm$ f min., the upper sign being used if stresses are in the opposite direction, and the lower if they are in the same.

$$\phi = \text{ratio } \frac{f \text{ min.}}{f \text{ max.}}$$

μ = primitive strength, being the greatest stress which the material can bear for an indefinite number of repetitions, the load being alternately applied and removed.

v = vibration strength, being the greatest stress which the material can bear an indefinite number of times when alternately applied in one sense but of equal intensity.

c = an unknown coefficient.

n = a constant for various materials.

The table gives the results of some tests upon various metals, with

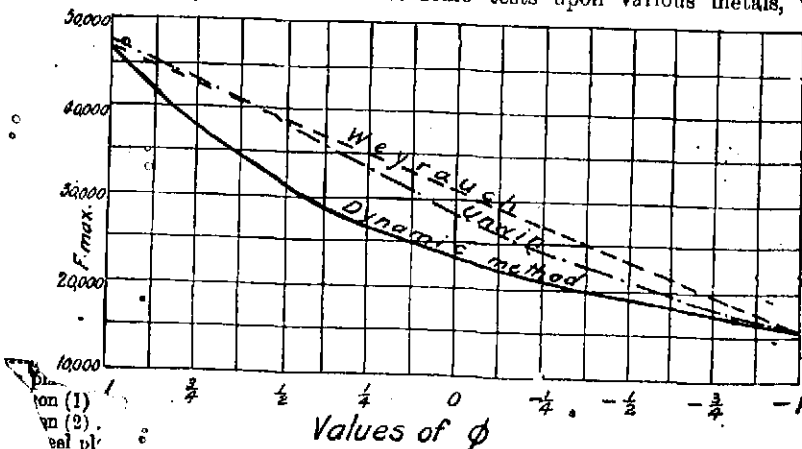


FIG. 89.

should be very carefully compared by the reader. They are the latest obtainable at the time of writing.

Now

$$f_{\max} = \frac{\Delta}{2} + \sqrt{f^2 - n\Delta f} \quad (43)$$

According to an authority (Herr Lunhardt)

$$f_{\max} = \mu \left(1 + \frac{f - a}{\mu} \phi \right) \quad (44)$$

This equation may, however, be simplified into

$$f_{\max} = \frac{2}{3} f \left(1 + \frac{\phi}{2} \right)$$

Equations 43 and 44 are plotted in fig. 89.

Now we have before spoken of factor of safety, both for live and dead loads. Say, however, we have a mixed load, such as a railway bridge.

Train = live load

Bridge itself = dead load.

Range of stress considered (this will be

For testing materials we have various machines, but it would be unnecessary to describe them here. They are, however, constructed to give diagrams to show us what goes on in the material.

A typical example is given in fig. 90, wherein we have load in tons vertically and extension in inches horizontal.

Now OA_1 shows a very small extension resulting from a load of about 18 tons, and directly proportional to the stress (they always correspond to some particular value of "E"). If allowed to, the test piece will return to its original condition apparently none the worse. The portion $A_1 A_2$ is not always present; it shows the strain increasing at a much greater rate than the stress, and during this period the material gets a small amount of permanent set. A_2 is the breakdown point. Very often we have a release of stress $A_2 B$ after the yield point, and then a rapid elongation with very little load. $C_1 - C_2$ is what is called the period of uniform flow, which will to a certain extent depend upon the time during which the load was applied. C_2 is the point of maximum or breaking load, while $C_2 D$ shows a decrease of stress due to breakdown. D is the final load. Now wrought iron and mild steel both possess what we call *plasticity*, cast iron and hard steels do not. When specifying metal we usually demand a certain elongation and contraction of area over a space of about 8 in. to 10 in., by which the plasticity is measured.

As shown by the following short specifications.

1. *Steel Castings*.—To have a tensile strength of not less than 28 tons per sq. in. with an elongation of 23 per cent. in 2 in. Bars 1 sq. in. in sectional area, to bend cold over a radius of 1½ in. to 90° at 28 tons tensile, up to 60° at 35 tons tensile.

2. *Castings* to be dropped on to a hard floor through a distance of at least 2 ft.

3. *Steel Forgings*.—To have a tensile strength of not less than 28 tons per sq. in., with an elongation of 30 per cent. in 2 in., bar 1 sq. in. in sectional area, to bend 180° over a ¼-in. radius, when cold.

4. *Cast Iron*.—To have a tensile strength of 9 tons per sq. in., and a transverse breaking load on a bar 1 sq. in. in area of 2000 lbs. over a span of 12 in. (We shall return to this subject under "Materials of Construction.")

We have the following information regarding the behaviour of metals under test as the result of countless experiments.

1. Steel of 19.5 tons tensile would stand an indefinite number of pulls and pushes of +7 or -7 tons, of +13 and 0 tons, and of +19 and 10½ tons.

2. By small repeated stresses below the breaking point a piece of iron may be broken.

3. The range of stress usually denotes the limit of reversals for rupture, which range decreases as the max. stress increases.

4. When we raise the tensional elastic limit we decrease the compressional (and vice versa); after a few reversals of stress it gets a fixed limit. This is

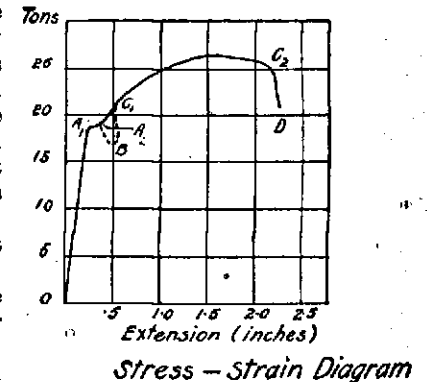


FIG. 90.

$$\therefore A \times 15 = 10 \times 10 = A = \frac{10 \times 10}{15} = \frac{20}{3} = 6.666 \text{ tons.}$$

The lever has numerous practical applications for engineers, although more especially to the design of machines than to structures.

However, a very prominent application is the ordinary hand force pump, as in fig. 97, where A is the force exerted by the man (usually about 120 lbs.) and B the pump piston, which in this case will exert a pressure on the water in the pump* barrel of $3 \times 120 \text{ lbs.} = 360.0 \text{ lbs.}$, which if it was acting on a plunger 4 sq. in. in sectional area $= \frac{360}{4} = 90 \text{ lbs. per sq. in. pressure,}$ enough to force the water to about 47 feet high.

In fact, the various practical forms are too numerous to mention. A claw hammer pulling out a nail or a pair of scissors are applications of simple levers. By suitable arrangements enormous power can be obtained with a comparatively small force applied. One of the best known lever forms to the engineer is perhaps the crank on an engine or pump to turn rectilinear motion into circular; for instance, if on the pin we have a force of 400 lbs. and the distance from centre of crank pin to centre of shaft is 1 ft., we exert a force of 400 lb.-ft., which is called a torque.

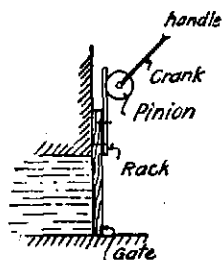


FIG. 99.

Now gear wheels, racks, and pinions are all levers, designed to give a very high mechanical advantage. For instance, we have a sluice gate to raise. It has a vertical rack and horizontal pinion, which has a crank on it 2 ft. long, as in fig. 99. A man exerts 120 lbs. pressure on the handle of the crank $= 120 \times 2 = 240 \text{ lb.-ft.}$, the pinion is 1 ft. in diameter $= 6 \text{ in.} = .5 \text{ ft.}$ in radius, $\therefore$ at the circumference of the pinion the pressure exerted is

$$\frac{240}{.5} = 480 \text{ lbs.}$$

Say, however, instead of having the crank on the same shaft as the pinion, there was another shaft (called a countershaft) gearing into the pinion by means of a smaller pinion. This smaller one we will say is 6 in. in diameter, the larger one 2 ft. in diameter; at the end of the crank, which is 2 ft. - 0 long, a man applies a force of, say, 100 lbs.; this $\times 2 = 200 \text{ lb.-ft.}$ applied to the pinion 12 in. diameter $= 6 \text{ in.} = .5 \text{ ft.}$ radius we have a force of $\frac{200}{.5} = 400 \text{ lbs.}$ Now we have a small pinion of 6 in. diameter, gearing

into a large one of 24 in. diameter. $\therefore \frac{24}{6} \times 400 = 1600$. $\therefore$ the man can raise a sluice weighing 1600 lbs., a mechanical advantage of 16. The screw is another application of the lever. We speak of screws having so many threads per in., and having a certain pitch. A screw with a $\frac{1}{4}$ -in. pitch has 4 threads to 1 in., and one of $\frac{1}{8}$ pitch 8 threads to 1 in., etc. Also, if we revolve a screw having a $\frac{1}{4}$ -in. pitch in a fixed nut just one revolution, it will go in or come out of the nut $\frac{1}{4}$ of an in. according to the direction of rotation, and whether it is a left-handed or right-handed screw. The difference between a right-hand thread (which is usually used) and a left-hand one is this. Take a nut and bolt, put the nut on the bolt and turn it in a clockwise

* The pump lever is assumed to be 3 ft. long.

direction. If the nut goes on the bolt it is a right-handed screw, if it comes off it is a left-handed one. The threads usually employed on bolts, nuts, etc., for fastening purposes is the Whitworth Standard; if for the transmission of power, such as lathes, sluice gates, etc., the square thread is usually employed.

For instance, in our example of a sluice gate before, if we had, instead of the gearing, rack, and pinion described, decided to work it by a screw motion, we should have made it as in fig. 100, having a fixed nut on the gate at A, in which worked a large square-thread screw of $\frac{1}{2}$ " pitch, running on the frame at B. There is a hand-wheel at C of 2' in diameter; then a man applies a force of 100 lbs. at the wheel, the wheel is $2 \times 3.1416 \text{ ft.}$ in circumference $= 6.2832 = 628.32 \text{ lbs.} = \text{ft.}$ in every revolution. But in every revolution the gate only moves $\frac{1}{2}$ ". $\therefore$ as $\frac{1}{2}" = \frac{1}{24}$ of a foot, $628.32 \div \frac{1}{24} = 628.32 \times 24 = 15,079 \text{ lbs.}$ $\therefore$ he could by the above means (neglecting friction) lift a sluice gate of 15,079 lbs. in weight, or nearly 7 tons. In the same way he could by applying the pressure of 100 lbs. to a press (such as a letter press) having the same velocity ratio, cause a pressure of nearly 7 tons. We shall treat on sluices later.

In the same way, if we have a shaft transmitting 100 h.p., and on it a pulley 2 ft. in diameter, driving by means of a pulley 4 ft. in diameter a countershaft, then that countershaft exerts twice the torque of the main shaft, although the horse-power transmitted remains the same, because horse-power $= \text{work} \times \text{time}$, and of course the countershaft only travels half as fast as the live shaft.

In this chapter we have only really touched upon the elements of mechanics which concern the reader, and which will help him to understand what follows. There is much more to be learnt, and matter which can only be fully grasped by working out the practical examples given in the various text-books before mentioned. We shall now proceed to apply our simple reasoning to practical questions in reference to beams, struts, and pillars, and the stability of walls. Before closing, however, we will describe the 300-ton testing machine at Birmingham University.*—

"The Civil Engineering Department of the Birmingham University is now possessed of one of the largest testing machines ever constructed. The leading points of the machine are its high capacity and the size of the specimens it is possible to test. Not only can small specimens be tested, but complete members themselves, which renders the test more valuable from every point of view. The necessity for such tests of complete members of a built-up structure has long been emphasised by engineers in all parts of the world, and without doubt will add considerably to the engineer's knowledge of the value of bracings and lattices, etc., as stiffening agencies, in the construction of compression members.

"The machine will also test ropes, chains, or bar specimens of the usual prepared or unprepared type, in tension up to a maximum stretched length of 33 ft. 6 in.; columns or stanchions up to a maximum length of 30 ft. can be tested in compression. The transverse test has received special attention, with the result that beams, built-up girders, or even roof trusses complete can be tested up to the maximum load applied centrally between abutments set to any span up to 20 ft. apart. The maximum depth of the beam or built-up girder in the transverse test is 30 ft.

* Mechanical World.

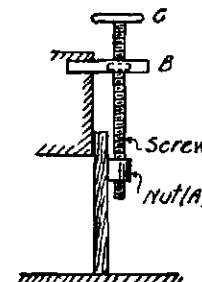


FIG. 100.

"The machine consists of a massive cast-iron frame, consisting of the bed, which has arranged at each end two vertical standards between which at the top are spanned two long horizontal columns acting as compression struts. The frame is so designed that it enables the operator to have a clear view of the specimen at each side for its full length, also the top of the specimen. This point is of the greatest importance when testing built-up members in compression, as it enables the buckling tendency of the lattices and bracings to be kept under close observation throughout the test. At the right hand of the frame is the hydraulic cylinder and ram for applying the strain, the weighing levers and steelyard being placed at the opposite end.

"The cylinder and ram are double-acting and arranged to suit a hydraulic pressure of 1000 lbs. per square inch, a pressure of 85 lbs. per square inch being used both for returning the ram to its normal position after a test and for applying test loads of small capacity. The large ram has a stroke of 5 ft. 6 in.; it is hollow, and works over a smaller subsidiary and stationary ram.

"For the straining stroke the hydraulic pressure is admitted behind the main ram, the return stroke being obtained by the pressure passing through the subsidiary ram. The weighing or recording mechanism consists of two levers, by means of which the test load is communicated to the steelyard.

"The main lever, which is of the bell-crank type, is a substantial steel casting fitted with hardened-steel knife-edges. The load and fulcrum knife-edges both conform to the Board of Trade Regulations, being 60 in. long, the large distance between them—i.e. 12 in.—enabling great sensitiveness to be attained. The lever is so disposed as regards the frame that the fulcrum knife-edge engages with a hardened-steel bearing fitted into a recess placed in the lower part of the bed. The second lever is fulcrumed upon a column standing upon and bolted to the bed, and at its outer end is connected to the rear knife-edge of the steelyard by means of connecting links having hardened-steel bearings.

"The hardened-steel knife-edges are ground perfectly lineable, and in order to prevent side friction are provided with hardened-steel ribs. The steelyard and poise are special to this machine, as the object has been to dispense with loose weights, which are always objectionable when conducting scientific tests. In this steelyard there are seven poises, each of which when moved to the full length of the steelyard represents 100,000 lbs. load upon the specimen, and the operator standing at either end of the steelyard has control over the poises. In order to obtain the best testing results, the strain may be kept on the specimen and the steelyard be constantly in equilibrium throughout the test until fracture occurs.

"The specimen in tension is secured at one end of a travelling crosshead, which is propelled along machined tracks upon the bed by means of gearing, operated by a handle at each side. When in position to suit the specimen, the crosshead is locked to four steel racks which slide in machined grooves in the bed and the two upper columns respectively. The four racks are connected to the head of the main ram, and partake of its motion, thus applying the strain to the specimen. The opposite end of the tension specimen is secured in a weighing crosshead which is suspended by means of a knife-edge resting in hardened-steel bearings upon the upper columns of the frame. This crosshead is connected by means of four mild-steel tension rods to a second crosshead at the other end of the bed, which is suspended, the whole weighing system of two crossheads and four tension rods thus

floating upon knife-edges,—this arrangement ensuring the greatest possible sensitiveness to be obtained.

"The floating frame is connected to the main lever by means of massive steel forks fitted with a hardened-steel bearing block which engages with the load knife-edge of the lever. Compression specimens are tested between platens attached to the travelling crosshead and the front face of the cross beam used for transverse tests. For the longest specimen the cross beam is removed and the column allowed to bear against the platen, which is then fitted into the crosshead. Two pairs of platens for column tests are provided, one pair having fixed surfaces for tests with fixed ends. The second pair of platens are Avery's patent swivelling type, enabling "free end" compression to be made.

"Transverse tests are made by placing the beam or girder to be tested across the two end pressure feet of the cross beam of the tester, and then causing the central presser foot which is bolted to the front face of the travelling crosshead to bear against the middle of the beam. The two presser feet upon the cross beam are set to the required space by means of hand-wheels at each end of the beam.

"The machine is provided with a staircase and platform upon which the operator stands to manipulate the steelyard poises, having at the same time under his control the valves by which the strain is applied. From this position, also, the operator has a complete view of the specimen while being tested. An autographic stress-strain recorder is attached to the steelyard column, and by connection to the specimen and the gearing by which the poises are propelled respectively enables stress-strain diagrams to be taken."*

The Mechanical Properties of Metals.

We have already discussed the term "elastic limit," that is, the true elastic limit. In practical testing of materials we have what is known as a commercial elastic limit, usually slightly higher than the true one. It is the stress at the yield point. The necessity of considering the question is because perfect elasticity is liable to fail just below the yield point, due, it is stated, to small portions of the material reaching the breakdown point before others. Hence we find the more ductile materials with a more clearly defined yield point.

In no material will yield take place simultaneously in the whole mass. For instance, in unmachined specimens in the test machine the skin flies off first. The importance of ductility in materials used in construction cannot be overrated, because it will tend to avoid failure due to a sudden or unexpected high local stress, due perhaps to a slight error in design.

It is in order to rely upon obtaining such ductility that the standard practice of limiting the limits of ultimate tensile strength and also the specifying the percentage of elongation is adhered to by engineers. The contraction of area is not so important. The length over which this contraction takes place must be rigidly set forth. 8 in. is usual—a practice which fails, however, to give comparative results of much value, due to most of the contraction being on a small length of about 2 in.

Hence the thicker bar shows the highest percentage of elongation. This could be obviated by the consideration of the general extension before maximum load, but this cannot be accurately measured after fracture. It is,

* *Mechanical World.*

however, possible to compare the ductilities of two bars of different diameter by making the length of elongation proportional to the $\sqrt{A}$ of the area (or diameter in round bars), in which case the length is made equal to $11.3 \sqrt{A}$. The cost of machining these special test pieces must, however, be very high. It is not usual to consider the reduced sectional area in taking measurements, the original being generally adhered to. This practice is based on the assumption of the metal being perfectly plastic, and the percentage of contraction of area being the same (when reckoned on the original area) as the percentage of the elongation reckoned on the final length, an assumption which does not always hold good. The shape of the test piece has also an important bearing on the results obtained.

Let us briefly consider various metals while under test.

1. Cast iron, which is a brittle metal and admits of very little elongation. The ultimate strength is rarely much over 10 tons per sq. in., and the value of E will be greatest when measured on the first ton of load. The elastic limit is very low. There may, however, be a permanent set due to tension, caused by this tension overcoming a certain initial stress in the metal when it cooled after casting. It will be removed by a heavy tensile load. On the other hand, we can have as much as 50 tons for the ultimate compressive stress (per sq. in.).

Wrought iron, on the other hand, by reason of its fibrous structure, gives a much higher value for tensile strength. Especially so is this the case in the direction of the fibres. It may be here stated that the fibrous structure (which is due to the rolling) is only a property seen by the casual observer; that is to say, the metal viewed microscopically is highly crystalline. Mild steel is a much more important metal to the engineer, being more ductile and uniform. There is no fibre, due to the bars being rolled from a cast ingot; hence the metal is more homogeneous, a fact evinced by the wear of the modern steel railway metals gradually to a smooth surface as compared with the old iron rails, which, after a certain amount of wear, used to flake off at the edges. The steel used in constructional work has generally less than one-half per cent. of carbon. It is usually made on the open hearth process. Having pointed out what occurs to metals under tensile stress, we proceed to see what occurs under a compressive stress, although compression tests are rarely made on account of being more difficult than tensile ones, and the values of E are practically the same in both cases. It is interesting to observe, however, how fracture does actually take place under compressive loads.

Some materials simply expand laterally, such as aluminium, the increasing area of cross section thereby preventing accurate results being tabulated. Again, metals not so plastic as aluminium will, during this lateral expansion, crack; this is specially noticeable in wrought iron. Hard and brittle materials usually shear across oblique planes, but this material is not in pure shear, because, in addition to the tangential or shear stress, we also have a normal compressive stress. The normal of the surfaces along which such shear takes place is of course inclined at a certain angle to the axis of direct compression, and the intensity of this shear stress is greatest when this angle (θ) = 45° , and is about 55° in cast iron. This angle will, however, vary according to what the test piece is bedded on in the machine. When the bedding material is such a soft one as lead it will allow of a certain amount of flow, which in turn gives greater freedom for lateral tension in the test piece, and consequent fracture at a lower load. Failure by shear, however, is by no

means confined to compressive loads, as may be supposed. It is a generally accepted fact now that, owing to ductile metals drawing out locally to a waist under test, final failure takes place, due to tangential component stress, because the fracture is often found to be oblique (about 45°) to the axis of pull; while in the harder materials the shape is generally that of a truncated cone, due to central core of the metal being harder, and tearing, and the skin shearing.

CHAPTER VIII.

BEAMS.

In the previous chapter the case of a beam loaded at different points over a certain space was considered. The object of referring to a beam was to apply the principle of moments to find what we termed the *reactions* on the supports. But the loads on this beam and any other beam set up stresses in the beam itself, and it is these which will be investigated.

In the present chapter any piece of a structure lying across two supports, whether loaded or not, is called a beam. A floor joist is a beam, likewise a girder. A lattice girder of a railway bridge is a trussed beam. The tubes of the Britannia Bridge are all beams, although, of course, of a very elaborate kind; in fact, large built-up steel beams are usually called *girders*.

When we load a beam it undergoes what is called a *bending moment*. In order that the beam may be safe there must also be a *resisting moment* equal to the bending moment. It is the material of the beam which supplies the resisting moment (to breaking), which, according to the factor of safety employed, may be five or six times, etc., the bending moment. In the sequel, BM stands for bending moment and RM for resisting moment.

Again, we may have a beam not supported at both ends but supported, and *fixed* at one end only; it is then called a *cantilever* (or *semi-beam*). Beams may also be fixed at both ends. The various calculations in them then become different. Now the bending moment of a beam at any section is measured by the moment about that section of the applied forces tending to turn the portion of the beam on the right or left hand of the section. For instance, in fig. 101 we have a beam over a span of 12 ft. carrying 10 tons at the centre. The BM at the middle will be found by considering the forces on the right- or left-hand side. We obviously have 2 equal reactions

of 5 tons each $\therefore 5 \times \frac{12}{2} = 30$ ton-feet. That is the bending moment.

But we may want the bending moment at a point A, a point 3 ft. from the left-hand support. Here we have a BM of $5 \times 3 = 15$ ton ft. Again, if we consider the right-hand portion, we have two quantities, viz. $(5 \times 9) - (10 \times 3)$, the force of 5 tons being upwards and 10 tons downwards. $\therefore (5 \times 9) - (10 \times 3) = 45 - 30 = 15$ ton-ft. as obtained before.

In fact, to get the BM of any beam—

1. Calculate the reactions of the supports.
2. Calculate the algebraic sum of the moments about the section of all the force acting,
 - (a) either on the right-hand portion,
 - (b) or left hand portion of the beam.

Now, as well as a bending moment in a beam, we have what is called shearing force,—that is, the tendency of one portion to slide over the other. To calculate the total shearing force at any section of a beam, take the algebraic sum of all the applied forces acting either—

- (a) on the right-hand side of the beam,
- (b) on the left-hand side of the beam.

For instance, in fig. 102 we have a beam loaded as shown. We want to find the shear at point C. Find the reaction on each support in the usual way: their values are—

$$\begin{aligned} P &= 4.25 \text{ tons.} \\ Q &= 4.75 \text{ tons.} \end{aligned}$$

Therefore on the left-hand portion we have an upward force (the reaction) of 4.25 and a downward one of 2 tons. $\therefore$ shear $= 4.25 - 2 = 2.25$ tons. Again, reaction at Q $= 4.75$, and we have a downward force in this portion of 7 tons. $\therefore 4.75 - 7 = 2.25$ tons shear same as before.

This is what is called a *positive* shear, because the preponderance of load on the right-hand side tends to cause that portion to slide down over the other in a clockwise direction. If the reverse be the case, there would be a *negative* shear.

In any beam uniformly or centrally loaded we have the greatest BM at

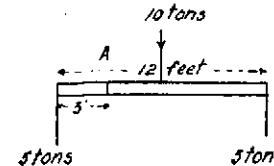


FIG. 101.

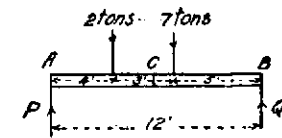


FIG. 102.

the centre and the greatest shear at the supports, and nil at the centre. But in a cantilever the BM is greatest at the supports, for this reason that we find in built-up girders more web stiffeners at the ends and more boom plates at the centre, because the booms take the BM and the webs or lattice work the shear. Any observant engineering pupil will have seen this.

Beams are made of various sections, rectangular and I beams being the most common. The latter are usually steel; they were some years ago frequently cast iron.

Now the strength of any beam is proportional to—

1. Breadth, depth squared,
2. Inversely as the span,

$$= \frac{BD^2}{L} \quad (53)$$

whereas in stiffeners it is proportional to—

1. Breadth, depth cubed,
2. Inversely as length cubed,

$$= \frac{BD^3}{L^3} \quad (54)$$

B and D measured in inches and L in feet.

Taking the two most common forms of beams, we have for one of I section—

Case VIII. Beam supported and loaded (fig. 110).—With a number of loads in various positions, first of all find the reactions on the supports R_1 and R_2 (vide Chap. VII.).

$$\left. \begin{aligned} \text{Then BM at any point } Y &= R_2 y - W_3 [y - (L - x_3)] \dots \\ \text{Equivalent distributed load} &= \frac{\text{Max. BM (in ft. tons)} \times 8}{L} \end{aligned} \right\} \quad (64)^*$$

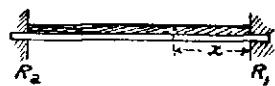


FIG. 111.

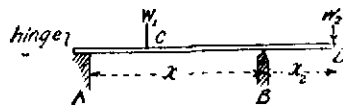


FIG. 112.

Case IX. Uniformly loaded beam (supported) (fig. 111).

W = total load

w = load per foot-run.

$$\left. \begin{aligned} \text{Max. BM occurring at centre} &= \frac{WL}{8} \text{ or } \frac{1}{8} wL^2 \dots \\ \text{,, at any point } x &= \frac{wx}{2} (L - x) \dots \\ \text{Shear at any point } x &= w \left(\frac{L}{2} - x \right) \dots \\ \text{Max. shear, which occurs at supports} &= \frac{W}{2} \dots \end{aligned} \right\} \quad (65)$$

Shear at any point x is positive when x is greater than $\frac{L}{2}$ and negative when less than $\frac{L}{2}$. It is nil at the centre.

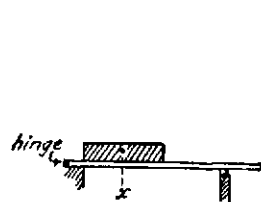


FIG. 113.

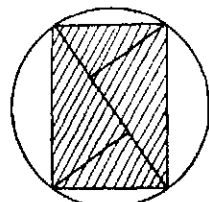


FIG. 114.

Case X. (fig. 112).

$$\text{First find } R_2 = \frac{(W_1 \times CA) + (W_2 \times DA)}{BA} \quad (66)$$

Then to find any BM between A and B we take R_2 and multiply by distance from R_2 to the point in question.†

Case XI. same as Case X. (fig. 113).—Take the total load W as concentrated at x , neglecting W_2 .

* L = distance between supports, as in fig. 109.

† R_2 = reaction at A.

Now the strongest beam will have a breadth of 5 to a depth of 7. When we have a log of timber and want to get the strongest beam out of it we proceed as in fig. 114. Draw a diameter and divide it into 3 parts. Erect perpendiculars thereon, and construct the parallelogram as shown. If, however, we divide the diameter into 4 parts, we have the stiffest beam. To test the above construction we have the formula for breaking load on any beam

$$= \frac{2cbd^2}{L} = w \quad (67)$$

b = breadth and d = depth in inches.

L = span in ft. and w = distributed load (total), c being a coefficient as follows:—

4 for northern pine	}	18 Wrought iron
5 Pitch pine		
3.5 Spruce or fir		20 Steel

If the load were central we would omit the 2 from the top line of the fraction. w is in cwt. always when using Formula 67. All beams, however, must, when designed as above, be incapable of any lateral movement.

When any beam has intermediate supports the two end bays should only be 75 of the others; this is because the middle supports cause the beam to become fixed.

The following table from Hurst's tables may be useful in the design of beams. It gives the portion of the load on each support of a uniformly loaded beam supported at several equidistant points.

Let W = load between each pair of supports.

Number of each Support.										
No. of supports.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
2	1/2	1/2								
3	1/3	1/3	1/3							
4	1/4	1/4	1/4	1/4						
5	1/5	1/5	1/5	1/5	1/5					
6	1/6	1/6	1/6	1/6	1/6	1/6				
7	1/7	1/7	1/7	1/7	1/7	1/7	1/7			
8	1/8	1/8	1/8	1/8	1/8	1/8	1/8	1/8		
9	1/9	1/9	1/9	1/9	1/9	1/9	1/9	1/9	1/9	
10	1/10	1/10	1/10	1/10	1/10	1/10	1/10	1/10	1/10	1/10

Note.—If the beams were perfectly rigid, all the supports except the end ones would sustain W , and the end ones would each sustain $\frac{1}{2}W$. Now considering a beam fixed at one end and supported at the other, we have what are called points of contraflexure in that beam; that is to say, at a certain point, instead of the tension being in the bottom of the beam and the compression in the upper parts, as usual, the reverse occurs. The points of change are the points of contraflexure. They can be found with precision as follows:—

r = allowable tensile stress in lbs. per sq. in., for wrought iron 5 tons, for cast iron $1\frac{1}{2}$ tons, and steel $6\frac{1}{2}$ tons.

If A = area of top flange in square inches, and D = depth (in.), then the resisting moment of that beam is

$$F \times a \times d \quad (55)$$

and of a rectangular beam

$$F \times b \times d^2 \quad (56)$$

For instance, we have an $18'' \times 7''$ I beam. What is its resisting moment?

Assume the top flange to be 7.5 sq. in. in sectional area, and A , which is what is called the effective depth, and measured from the centre of the flanges, = 17', and F , 5 tons. Then $5 \times 7.5 \times 17 = 637.5$ in.-tons resisting moment. If then we have a BM due to load of 100 in.-tons, the beam will stand a load of 6.375 tons.

In tables of properties of I beams, however, we often see what is called the modulus of section. This is the number by which the safe stress must be multiplied to give us the RM.

In the case of the $18'' \times 7''$ beam the section modulus is 127.6; this multiplied by the 5 tons safe stress = 638.0 only, .5 difference.

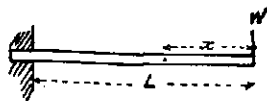


FIG. 103.

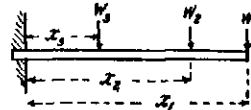


FIG. 104.

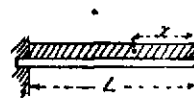


FIG. 105.

For a beam of circular section (not usually employed) we have a RM of

$$\frac{F\pi R^3}{4} \quad (57)$$

R = radius of beam in inches.

We will have observed that makers of I beams usually make the flanges equal in area: this is because the steel at present on the market is assumed to be equal in tensile to compressive strength. Now cast iron has only a tensile strength of about $\frac{1}{4}$ that in compression. Therefore any observer of cast-iron beams will observe them to have a large bottom flange, which always takes the tension, and a small top flange for the compression. In the case of cast-iron cantilevers the opposite occurs, as the top flange then takes all the tension and the bottom one the compression. Casting beams so in order to save metal has been deemed a bad practice, owing to contraction in cooling taking place unequally. This has caused many CI beams to fail, and is one (together with the fact that standard-sized steel joists are cheaper) of the reasons for their disuse.

We shall now discuss the various formulæ in reference to beams which occur in practice.

Case I. A cantilever (fig. 103).

$$\left. \begin{array}{l} 1. \text{ Bending moment at end} = WL \\ 2. \text{ " " any point } x = Wx \\ \text{Shear everywhere} = W \\ \text{Reaction at support} = W \end{array} \right\} \quad (58, 59)$$

Case II. Cantilever variously loaded (fig. 104).

$$\left. \begin{array}{l} \text{Bending moment at support} = (w_1x_1) + (w_2x_2) + (w_3x_3) \\ \text{Maximum shear (occurs at support)} = w_1 + w_2 + w_3 \dots \\ \text{Shear at any point} = \text{No. of weights behind } W, \text{ viz. towards the end} \end{array} \right\} \quad (60)$$

Case III. Uniformly loaded cantilever (fig. 105).

$$\left. \begin{array}{l} w = \text{weight per foot-run} \\ W = \text{total load} = wL \\ \text{Bending moment at end} = \frac{1}{2}wL^2 \\ \text{" " any point } x = \frac{wx^2}{2} \\ \text{Shear at end} = W \\ \text{" " any point} = wx \end{array} \right\} \quad (61)$$



FIG. 106.

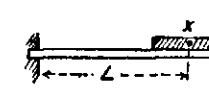


FIG. 107.

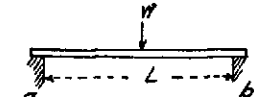


FIG. 108.

Case IV. Combine Cases I. and III. treating separately (fig. 106).

Case V. Treat as in Case I. assuming a single load at x (fig. 107).

Case VI. Centre loaded beam (fig. 108).

$$\left. \begin{array}{l} \text{Bending moment at centre} = \frac{WL}{4} \\ \text{Shear at centre} = \frac{W}{2} \\ \text{Reaction at supports } ab = \frac{W}{2} \end{array} \right\} \quad (62)$$

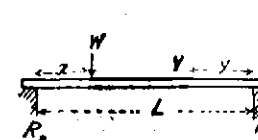


FIG. 109.

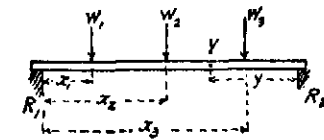


FIG. 110.

Case VII. Beam supported and loaded anywhere (fig. 109).—Max. BM occurs at point of W , viz. just below it. Its value is

$$\left. \begin{array}{l} \frac{Wx}{L}(L-x) \\ \text{bending moment at any point } Y = \frac{Wx}{L} \times y \\ \text{and at any point between } W \text{ and } R_2 = \frac{W(L-x)}{L}L-y \\ \text{shear on right-hand portion} = R_2 \\ \text{" " left- " " } = R_1 \end{array} \right\} \quad (63)$$

(viz. to right or left of W) (vide Chap. V.)

1. Beams fixed at one end and supported at the other.—If S = span, point of contraflexure occurs at—

$$\begin{aligned} (a) & \text{ Under a uniform distributed load } .267 S \dots\dots\dots \\ (b) & \text{ „ „ central load } .273 S \dots\dots\dots \end{aligned} \quad (68)$$

2. Beam fixed at both ends.

$$\begin{aligned} (a) & \text{ Under a uniform distributed load } .211 S \dots\dots\dots \\ (b) & \text{ „ „ central load } .250 S \dots\dots\dots \end{aligned} \quad (69)$$

The distance is measured from the piers, as in fig. 115.

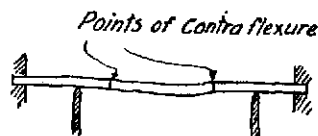


FIG. 115.



FIG. 116.—Flitched Beam.

Now we can also have what are termed flitched beams as in fig. 116, where we have two timber beams with an iron plate or flitch in between them. When cut, the two beams are placed heart side outwards and bolted, a process known as halving and reversing.

As regards the strength of such beams, we do not apply Equation 67, but taking b as the thickness of the whole when bolted, and t the thickness of the flitch (in inches), flitch being vertical, which is usually $\frac{1}{10}$, we have w = breaking weight at the middle in cwt.

$$= \frac{d^2}{L} (Cb + 30t) \dots\dots\dots (70)$$

L = span of beam in feet. C = coeff. as in Formula 3-0.

Another form of beam known as the trussed wood beam is used for a variety of purposes, such as small foot-bridges, temporary tramways, etc. It is a cheap and effective way of crossing small openings in out-of-the-way places, where the ironwork can usually be supplied by the village smith. We shall discuss the design later, but give diagram here to illustrate our reasoning. Take Case I., fig. 117.

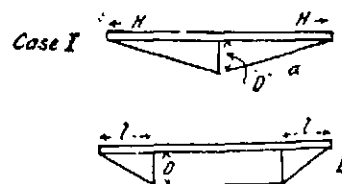


FIG. 117.

The applied load (which is assumed to be W tons, uniform) produces the reaction in the beam. They are two opposite thrusts

$$= H = \frac{WL}{8D} \dots\dots\dots (71)$$

which in turn sets up a strain in each of the tension rods

$$= S = \sqrt{H^2 + \frac{W^2}{16}} \dots\dots\dots (72)$$

Case II.—Here we have a central stay with a tension = H in Equation 71, and, to find S the tension in the ties,

$$S = \frac{H}{l} \sqrt{l^2 + D^2} \dots\dots\dots (73)$$

Therefore by the above calculations we can find the required section of area of the iron bars employed. The strut D may be calculated to bear a load equal to $\frac{1}{2}$ the load on the beams, but if the load were applied vertically over it, it should be capable of bearing that load.

By the use of Equation 67 we can compare two beams if necessary. Suppose we have a certain beam which is known (by tests or otherwise) to be capable of bearing w tons, cwt., or lbs. Its breadth is b , depth d , and span l , in the usual units. Again, we have a beam of breadth = B , depth = D , and span = L . What load will it bear = W (centrally applied)?

$$\text{Then} \quad W = w \frac{BD^2L}{bl^2l} \dots\dots\dots (74)$$

in all cases the small letters referring to the tested beam, and the large ones to our new one in question.

For sake of example take the following:—

A beam of teak of 1 inch square lying over a 12" span was found in a laboratory to break with a central load of 820 lbs. Find the breaking load of a bar of teak 3" x 3" over a 7-ft. span.

Here, in the first case, we have

$$b = 1", d = 1", l = 12", w = 820 \text{ lbs.}$$

In the second,

$$b = 3", d = 3", l = 7 \times 12 = 84" (= 7 \text{ ft.}).$$

Then by equation

$$W = w \frac{BD^2L}{bl^2l}$$

$$W = 820 \times \frac{3 \times 3 \times 3 \times 1}{1 \times 1 \times 1 \times 7} = 3162.8 \text{ lbs.}$$

We have up to the present been considering beams loaded with a steady uniform or central load, but many beams, when they serve the purpose of bridges (large or small), are subject to what we call a rolling load, which has a somewhat different effect. We also know the max. BM due to any load is always directly under that load. Take the case of the girders for a small hand travelling crane which one often sees constructed of timber in the yards of contractors or municipal authorities for lifting moderately heavy weights, and on a larger scale in big engineering and steel works for lifting 50 tons or more. Here we have two wheels as in fig. 118, moving towards the left at a common distance apart = c . It will be obvious to the reader that the maximum BM occurs just under the front wheel when travelling towards the centre, and under the back one when past that centre. Now find the BM for each wheel, similar to Case VIII., and construct the bending moment diagram as in fig. 119, the verticals being a distance apart = $\frac{1}{2}c$. The shear diagram is shown in the same figure. We do not propose here to treat upon BM and shear force diagrams; these are of no particular practical value in design, and are fully treated in all books on Mechanics.

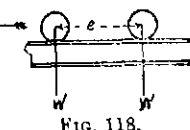


FIG. 118.

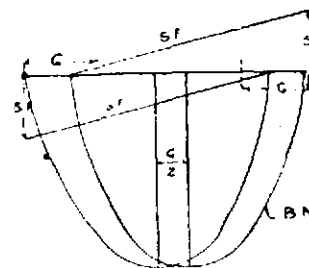


FIG. 119.

In the case, however, of a train moving over a bridge, the max. shear will occur just in *advance* of the front wheel, and it increases as the *square* of the length of the bridge covered by the train. In this case the BM is identical with Case IX.

Now when a beam is loaded we have pointed out that the upper fibres become compressed and the lower ones extended. This may be shown by having a beam of thin boards, as in fig. 120, where they will appear to overlap. There is a point in the beam where there is no such stress, that is, where the stresses change. It is called the *Neutral Axis*.



Fig. 120.—Loose Plank Beam.

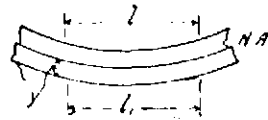


Fig. 121.

Then, if l = the length of any part of this neutral axis in a beam, and l_1 = length of under side of the fibres (fig. 121), the strain

$$= \frac{l - l_1}{l} \quad (75)$$

and if f is the stress on the beam producing this, we know that

$$E = \frac{\text{stress}}{\text{strain}} \quad \therefore E = \frac{f}{\frac{l - l_1}{l}} \quad (76)$$

Now there is a property in beams of much importance to the engineer called the moment of inertia. It is the product of the mass of every particle of any body $\times$ by the square of the distance of that particular particle from the neutral axis. $\therefore I$ = moment of inertia = a quantity in lbs., ton-feet, etc. And its value is

$$I = \sum mr^2 \quad (77)$$

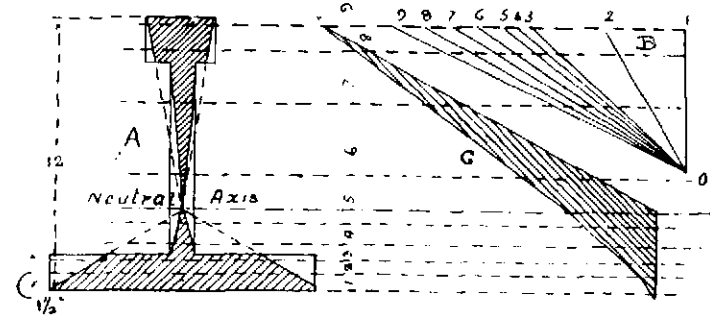
m = mass of every particle, r = radius of that particle, and $\sum$ the sign that all the values of $m \times r^2$ are added together, because we can take an indefinite number of them. r is called the radius of Gyration.

For finding moments of inertia for beam calculations it is usual to use tables to save time and trouble. Failing such tables, the following method may be used, besides being useful to know how to make such calculations for answering examination questions, etc.

The method of procedure is as follows:—

"The centre of gravity through which the neutral axis passes must first be obtained by calculation. Then the dotted lines in fig. 121a, from which the outline of the inertia area is obtained, are all produced upon the following method. Taking any point in the outline of the original section, project it on to the extreme edge on the same side of neutral axis, and perpendicular to the neutral axis, and from the edge draw to the intersection of centre line with neutral axis, then parallel with the neutral axis draw a line from the original point to cut the diagonal line drawn to centre. This intersection

will be the contraction of the point in the original outline to the new outline for inertia area. The second method is worked as follows. Divide up the section into any convenient number of parts, and find the area and centre of gravity of each part. Set down these areas 1 to 9 to scale as in fig. 121b, and draw 1-0 at right angles to 1-9, and equal to half of 1-9. Join 0 to the points 1 to 9 on the line. Draw horizontal lines from the centres of gravity of the various parts in fig. 121a, and parallel with the vectors in fig. 121b; across the corresponding spaces construct the inertia area (fig. 121c). The



Figs. 121 A, B, C.*

junction of the lines parallel with 1-0 and 0-9 will be a point on the neutral axis if the figure has been worked correctly. Using a planimeter to find the area, fig. 121a gives 1.431 sq. in., but as the scale is one-quarter full size, this area must be multiplied by $4^2 = 16$ to give the real area. Then the inertia area $a = 1.431 \times 16 = 22.9$ sq. in., the area of section $A = 32.5$ sq. in., and the moment of inertia $I = Aa = 32.5 \times 22.9 = 744.25$ in. units. Using the planimeter on fig. 121c, the area is found to be 1.42 sq. in., and the true area $= 1.42 \times 16 = 22.72$ sq. in. The moment of inertia will then be $I = Aa = 32.5 \times 22.72 = 738.4$ in. units; but had more divisions been taken on the section, this result would have been much closer to the first method. The first method is the more accurate, and it also permits of ready calculation of area as a check; while the second method is often quicker for use when the moment of inertia for irregular sections is required."

Now all beams are subject to a certain deflection when loaded. We have an equation which gives us that deflection; say we have a beam loaded with a load = W lb.

Then if E the modulus of Elasticity of the material of which the beam is made, and I the moment of inertia of that beam in inch-lbs., and l = span in inches, then the deflection of that beam in inches

$$= D = \frac{KWl^3}{EI} \quad (78)$$

K being a coefficient as follows:—

$$\left. \begin{array}{ll} \text{Uniformly loaded cantilever} & K = \frac{1}{8} \\ \text{End} & K = \frac{1}{3} \\ \text{Centre} & K = \frac{1}{48} \\ \text{Uniformly} & K = \frac{1}{7} \end{array} \right\} \quad (79)$$

* *Building World*.

Such a deflection must in no case be more than $\frac{1}{1000}$ of the span, under ordinary circumstances, under the actual working load.

Now we know that the BM of a beam should be equal to the resisting moment of that beam also; then in any beam under deflection of the radius of curvature of the deflection = ρ ,

$$BM - RM = \frac{E \times I}{\rho} \quad (80)$$

and the radius of curvature

$$\frac{BM \text{ or } RM}{EI} \quad (81)$$

and given f = the allowable tensile or compressive stress at any distance y from the neutral axis, then

$$f = \frac{M}{I} \times y \quad (82)$$

This equation may then be transposed to give I , M or y . When y is max. f is max., and the modulus of section spoken of before

$$= Z = \frac{I}{y} \quad (83)$$

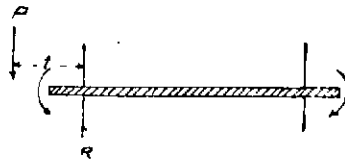


FIG. 122.

When, however, the beam section is not symmetrical, there will be two values of y and Z . These values always appear in tables of properties of beams.

Again, if we are given f max. then

$$M = Zf \text{ max.} \quad (84)$$

M being the bending moment. Formula 84 transposed will give

$$f_1 \text{ max.} = \frac{M}{Z} \quad (84a)$$

But the moment of resistance

$$= \frac{fI}{y} \quad (85)$$

and when we are considering the breaking load, f is called the modulus of Rupture.

Regarding the fixing of beams, our previous examples were confined to beams merely supported.

In practical work they are very often rigidly fixed, and this alters the calculations. In large bridges we often have one end fixed, and one end supported only on a series of rollers. This counteracts the effects of expansion. The result of fixing a beam is a bending moment in that beam constant all over it.

Referring to fig. 122, in order to keep the beam horizontal, we have a downward force P at a distance = l .

$$\therefore \text{the BM in question} = Pt \quad (86)$$

Here we have an upward force or reaction at the supports = R . Now we said this particular BM was constant all over the beam; but if the beam is loaded, we then have a bending moment in an opposite direction, due to the load. The fixing reduces this bending moment due to load to $\frac{1}{2}$ its original value, but it occurs at the centre and at each end also. We see a beam under these conditions in the figure 123, F and G being called the points of

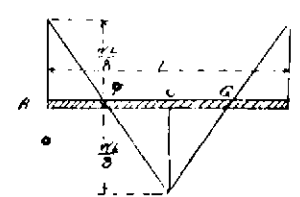


FIG. 123.

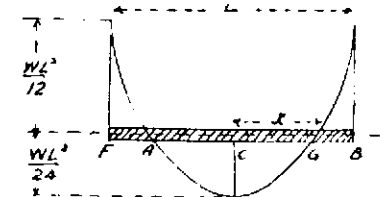


FIG. 124.

inflection. In big bridges it is these two points that rest on rollers to allow for expansion. In such cases the deflection, Equation 78, is reduced to $\frac{1}{4}$ of its value, so that the beam is now four times as stiff. When the load is distributed the conditions become altered.

We see by Case IX. that $BM = \frac{1}{8} wL^2$. Fig. 124 shows the curve of bending moment for a fixed uniformly loaded beam. Here we have $\frac{1}{8} wL^2$ broken up into $\frac{1}{12} wL^2$ and $\frac{1}{24} wL^2$. We know that the neutral axis of a beam usually lies on its centre. It is proportional to E . Say, however, E is different for tension and compression, as in cast iron. Then the neutral axis lies towards that side of the beam (the tension or compression) corresponding to which E is greatest. Sometimes we require to find the tensile and compressive stresses set up in a beam.

Given •

- M Bending moment of that beam,
- a_c = Area of compression flange (sq. in.),
- a_t = " " tension flange (sq. in.),
- H Mean depth (in.),

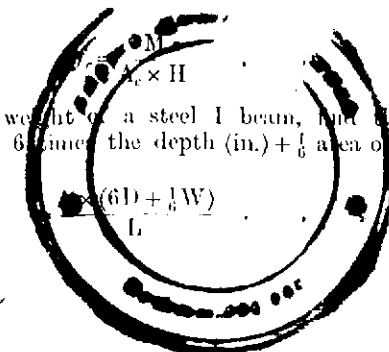
then the tensile stress (lbs. per sq. in. when BM. = in.-lb.) set up is equal to

$$f_t = \frac{M}{A_t \times H} \quad (87)$$

and the compressive stress

$$f_c = \frac{M}{A_c \times H} \quad (88)$$

To find the breaking weight of a steel I beam, find the area of the bottom flange in sq. in. $\times 6$ times the depth (in.) $+$ $\frac{1}{6}$ area of web in sq. in. $\div$ span in ft.



$$\frac{(6A_t + \frac{1}{6} A_w)}{L} \quad (89)$$

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If for cast iron, area of bottom flange $\times$ twice depth $\div$ span gives it

$$= \frac{A \times 2D}{L} \quad (90)$$

For the purpose of illustrating the reasoning of the foregoing chapter, let us take a few practical examples.

We have a beam to design in steel, cast iron, timber (flitched, if desired), or trussed.

Concrete steel beams will receive special consideration in Chapter XX.

Load	= 10 tons distributed.
Span	= 40 ft.
Reactions on each support are equal . . .	= 5 tons each.
Bending moment, max. at centre	= 50 ton ft.
Shear, max. at ends	= 5 tons.

Try a pitch-pine joist 10 in. $\times$ 14 in., using Formula 67.

$$W = \frac{2cbd^2}{L} = \frac{2 \times 5 \times 10 \times 14 \times 14}{40 \times 20} = \frac{49}{2} = 24.5 \text{ tons}$$

= factor of safety of 2.45, which is somewhat small.

Let us try the same beam, halved and flitched, using a 1-in. flitch plate. Using Equation 70 we have

$$\begin{aligned} W &= \frac{14 \times 14}{40} (3.3 \times 10 + 30 \times 1) \times 2 \\ &= \frac{49}{10} \times 63 \times 2 = 4.9 \times 126 \\ &= 617.4 \div 20 = 30.87 \end{aligned}$$

$\div 10$ = say a factor of safety of 3.

Say, however, we try a 16-in. $\times$ 6-in. $\times$ 62-lb. RS joist, using Equation 55, transposed to give ultimate resisting moment.

F = ultimate strength = 30 tons, say.

$$\therefore 30 \times 6 \times 15 = 2700 \text{ in.-tons} = \text{RM.}^*$$

$$\text{RM} = 12 \times 50 = 600 \text{ in.-tons.}$$

$$\therefore \frac{2700}{600} = 4.50$$

= a factor of safety of 4.5, which would do well.

Now, using Equations 87 and 88,

$$f_t = \frac{50 \times 12}{6 \times 16} = \frac{50 \times 12}{96} = \frac{50}{8} \text{ or } 6.25 \text{ tons per sq. in.}$$

the flanges being equal, $f_c = f_t$.

But say we had a cast-iron beam of the same depth, the tension flange is

* The area of the flanges is assumed = 6 sq. in. It will vary with the mean thickness of the flange according to the particular maker of the joist.

only capable of bearing $1\frac{1}{2}$ tons per sq. in. $\therefore$ it would have to be $6 \times 4.25 = 25.50$ sq. in. in sect. area, say 12 in. $\times$ 2 in., which would be thickened up at the centre to give 25.5 sq. in.

The top flange would be $\frac{1}{2}$ of this, say 5 sq. in. = 2.5 in. $\times$ 2 in., while the web would be the same thickness at the flange it adjoins.

Its form is shown in fig. 125. Stiffeners, called "Feathers," should be cast on the web to guard against lateral flexure. They should not extend quite up to the top flange. If they did they would probably fracture on cooling. Some engineers are adverse to the use of such feathers. Now the CI beam has a depth of 16 in.

Area of top flange = 5 sq. in.

" bottom flange = 25.5 sq. in.

Then, applying Formula 90, we have

$$\frac{2 \times 25.5 \times 2 \times 16}{40} = 40 \text{ tons breaking load, say}$$

= factor of safety of 4. $\therefore$ the CI beam would suit our purpose.

The latter questions in reference to the design of beams are most important to the engineer, that is, he should fully understand how to work such problems out; much time is now saved by tables, especially those of I beams and built-up steel joists, etc., where for a certain size we have the safe loads over any span in question. We shall refer to them again under constructional steelwork, and constructional carpentry and timber bridges for small spans, and give detailed drawings, etc. We shall now go to the consideration of the stresses which occur in pillars and stanchions and revolving shafting.

C.I. Beam

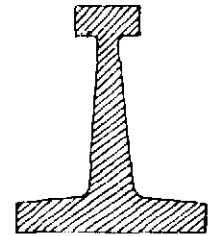


FIG. 125.

CHAPTER IX.

PILLARS AND STANCHIONS AND SHAFTING.

For the support of superincumbent loads various forms of stanchions are used, which range from the simple cast-iron column or brick pier to the most elaborate built-up steel stanchions.

With the common cylindrical cast-iron pillar, there is a great difference between the load which may be borne when it is *fixed* at both ends and when it is not so fixed. Such pillars usually are fixed, and have flat heads cast upon them, and we then have the two following important formulæ for the *breaking* loads:—

For solid cast-iron columns,

$$W = 42 \left(\frac{d^{3.5}}{L^{1.75}} \right) \quad (91)$$

and for hollow ones,

$$42 \left(\frac{d_1^{3.5} - d_2^{3.5}}{L^{1.75}} \right) \quad (92)$$

d being the outside diameter } in inches.
 d_1 " " inside " "
 d_2 " " " " "

W = breaking load.

L = length in feet, which must not exceed 30 times d .

The equations here have to be worked out by aid of logarithms, with which I assume the reader is familiar. But for the benefit of those who perhaps are not, there are other formulæ equally as reliable. Those given above are the results of the experiments of a Mr Hodgkinson, while the others, equally well known, are known as the Gordon Rankine formulæ.

As an example of Gordon's formula, No. 93 (*sup.*), let us take the case of a cast-iron hollow cylindrical pillar, 20 ft. high and 10" outside diameter, with metal $\frac{1}{2}$ " thick = 9" inside diameter. Then

$$W = \frac{36S}{1 + \frac{L^2}{400d^2}}$$

$$S = 10^2 - 9^2 \times .785 = 100 - 81 = 19 \times .785 = 14.91, \text{ say } 15$$

Then

$$\frac{36 \times 15}{1 + \left(\frac{20 \times 20 \times 12 \times 12}{400 \times 10 \times 10} \right)} = \frac{36 \times 15}{2.44}$$

= 221.3 = crushing load.

We use a factor of safety of 10 for such work, therefore we could place a safe load of 22 tons on 10" hollow pillar of cast iron $\frac{1}{2}$ " thick and 20 ft. in height. A 10" pillar would probably be made, however, more than $\frac{1}{2}$ " thick.

If such a case should occur that they cannot be treated as *fixed*, then only $\frac{1}{4}$ the above loads can be taken as the crushing loads.

Then we have the Rankine Gordon formulæ, which are known to give accurate practical results.

They are, where l = length in inches,

S = sectional area of the metal, in hollow pillars = $d_1^2 - d_2^2 \times .785$ (*vide* Chapter VI.).

d = external diameter in hollow pillars.

d = diameter in solid pillars.

d = diagonal if a hollow square.

d = shortest length if a cross section of unequal size.

Then for round, hollow pillars,

$$\text{Crushing load in tons} = W = \frac{36S}{1 + \frac{l^2}{400d^2}} \quad (93)$$

For hollow squares,

$$W = \frac{36S}{1 + \frac{l^2}{800d^2}} \quad (94)$$

For a cruciform section,

$$W = \frac{36S}{1 + \frac{l^2}{400d^2}} \quad (95)$$

but should the ends be hinged, in Equations (93) and (95) read $100d^2$ in place of $400d^2$, and in (94) $200d^2$ instead of $800d^2$. And in the three cases take a factor of safety of 10.

We can also have built-up steel tubes to form pillars. Ordinary steel we know will stand 6 tons per sq. in. under most loads, provided always the loading is axial, that is, acting truly along the axis of the tube. Built-up tubes have a breaking load of about 12 tons per sq. in. of section if standing alone.

When in series, however, this may be increased to 15 tons. Such tubes give way by collapsing, but in no case should the thickness of the plates be less than $\frac{1}{80}$ of the diameter.

Now this collapsing pressure is found by the formula

$$p = 806000 \frac{t^2}{dL} \quad (96)$$

where t = thickness in inches and d = diameter in inches if cylindrical and diagonal, if a square or rectangle, and L = length in feet.

Such tubes are much stiffened by rings of L or T iron, riveted on as in fig. 126. But we shall consider such work when dealing with steel and iron construction. Built joints are assumed in the formulæ. Now any column which if it partially gave way under load or during testing is bent and

Now in the previous chapter we referred to

$M = BM$ or RM ,
 $I =$ moment of inertia,
 $y =$ distance from outer fibre to neutral axis,
 $P_1 =$ intensity of outer fibre stress due to bending.

$$\text{Now} \quad \frac{M}{I} = \frac{P_1}{y} \quad (98)$$

Then by combining Equations 97 and 98 we have the following relation, that is

$$P\Delta\delta = \frac{P_1 I}{y} \text{ or } \frac{P_1}{P} = \frac{\delta y}{p^2} \quad (99)$$

$p =$ radius of gyration of the section.

Now the proof deflection of a beam varies as

$$\frac{l^2}{y} \text{ or } \delta = c \frac{l^2}{y} \text{ where } c \text{ is a constant} \quad (100)$$

so, substituting the value of δ in Equation 99, we have

$$\frac{P_1}{P} = c \frac{l^2}{p^2} \quad (101)$$

and adding unity to both sides of the equation,

$$\frac{P_1 + P}{P} = 1 + c \frac{l^2}{p^2} \quad (102)$$

Now $P_1 + P =$ sum of the stresses due to the load and to bending, and is equal to $f =$ the working strength of the material in compression; also $pA = P$. Then, by substituting, we have

$$P = \frac{fA}{1 + c \frac{l^2}{p^2}} \quad (103)$$

which is the same as Equation 93, but in slightly different symbols. That is to say, from the above reasoning we have deduced Gordon's formula. f being equal to 36,000 for wrought iron,

$c = \frac{1}{80000}$ for column with round ends,

$c = \frac{1}{80000}$ for column with square ends fixed.

The above deduction is, however, open to criticism, because it assumes the deflection of a column to be the same as that of a beam, whereas the deflection of a column when it comes to its proof stress is less than that of a beam compared under similar conditions.

In conclusion, it may be said that the Rankine Gordon formula is about as accurate as we can get. It errs on the side of safety, and is simple. We shall, as in the case of beam, return to pillars and stanchions in a more practical form when treating steel construction and constructional carpentry.

The following experiments were made at the Illinois University:—

$A =$ total area of plate in sq. ins.
 $P =$ total load in lbs. transmitted by it.
 $L =$ perpendicular distance in ins. from fracture line to centre of upward pressures on one half the area of the plate.

$$L = \begin{cases} .25 \times \text{length in ins. of a wall or bearing plate.} \\ .25 \times \text{side of square plate.} \\ .2187 \times \text{inscribed diameter of octagonal plate.} \\ .2122 \times \text{diameter of circular plate.} \end{cases}$$

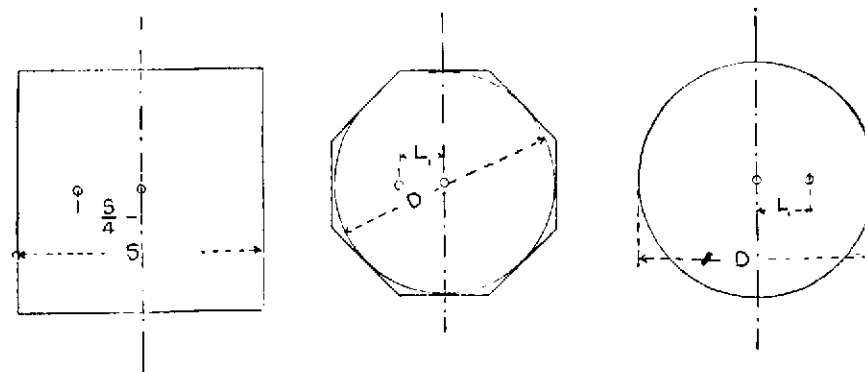


FIG. 130

Let $L' =$ perpendicular distance in ins. from fracture line to centre of downward pressures on one-half the area of the plate.

$$L' = \begin{cases} .25 \times \text{width of beam or girder on bearing plate.} \\ .25 \times \text{side of a solid square post or column.} \\ .2187 \times \text{diameter of solid octagonal post or column.} \\ .2122 \times \text{,, ,, cylindrical ,, ,,} \end{cases}$$

$R =$ radius of outer circle inscribed in cross section of hollow square or cylindrical column.

$r =$ radius of inner circle inscribed in hollow square or round column.

$f =$ maximum safe fibre stress in lbs. per sq. in.

$t =$ thickness of plates in ins. at centre under column.

Referring to fig. 130 we have diagrams of square, octagonal, and circular base plates, and for a square plate

$$t = \sqrt{\frac{3P(S - L')}{fS}} \quad (103a)$$

for an octagonal plate

$$t = \sqrt{\frac{3P(D - L')}{fD}} \quad (103b)$$

We will conclude our remarks on stanchions with a few theoretical considerations in reference to base plates. The foundation of a pillar or stanchion carrying a considerable load has always to cover a larger area

if it be carefully straightened and well hammered, the yield point will be raised and the ductility diminished. Such a column when retested will be found stronger than the original one. That is why engineers do *not* put too much faith in elastic limit, because if a bar be loaded before it goes in the testing machine, to say 20 tons per sq. in., the elastic limit at the test will be at *least* 20 tons. In the same way by compression the elastic limit may be lowered. There is no doubt many unsuspecting young engineers have been "had" in the above way during tests which they have been commissioned to witness by unscrupulous steel-makers.

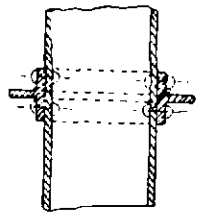
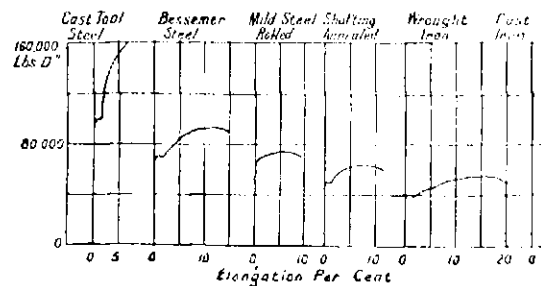
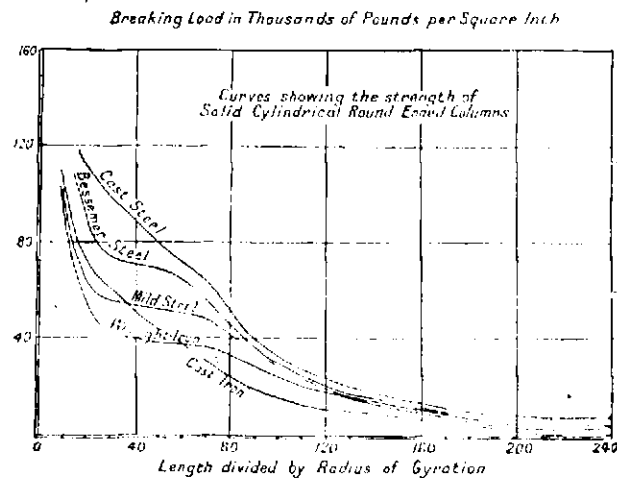


FIG. 126.

In order, however, that the pillars in question may have the yield point so raised the factor $\frac{l}{p}$ must be greater than 30 and less than 120. l = length in



FIGS. 127 AND 128.

inches, and p = radius of gyration also in inches. This is the reason that all members of a structure in compression should have a high elastic limit, and it was doubtless a knowledge of these facts that led the late Sir Benjamin Baker to specify a high elastic limit for the steel used in the compression members of the famous Forth Bridge.

According as the factor $\frac{l}{p}$ is less than, equal to, or greater than 60, so cast iron will be *stronger, equal in strength to, or weaker* than wrought iron for stanchions; and by working out the factor for any of our proposed sections, we can find out which is the best material to use. For instance, required a stanchion 8' 4" high, of sectional area 10" x 10". 8' 4" = 100" and the radius of gyration of a square = $\cdot 289 \times$ breadth.

$$\therefore \frac{100}{10 \times \cdot 289} = \frac{100}{2.89} = 34.5.$$

$\therefore$ 34.5 is less than 60. We should use cast iron.

Again, where $\frac{l}{p}$ is greater than 30 and less than 120, the circular form is the strongest shape.

Now fig. 127 shows the plotted results of a series of tests made in the author's presence on bars of various materials in a 10-ton Wicksteed testing machine in Dublin. The bars were $\frac{1}{2}$ " in diameter.

The object in view in carrying out the experiments was to determine constants to be used in formulæ relating to the design of solid, cylindrical, *round-ended* columns. The mode of testing was in all cases the same. Definite lengths of the bars were cut off and the ends rounded in a lathe, the bars being at the same time very carefully straightened; they were then placed vertically in the machine, the round ends bearing on slightly concave steel discs to enable the specimen to be adjusted. Now in fig. 128 we have the stress-strain diagrams corresponding to the tension tests.

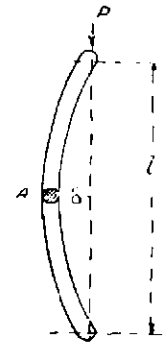


FIG. 129.

It will be noticed when $\frac{l}{p}$ is greater than 120 there is little difference between the breaking loads obtained on the various materials. For values beyond this limit the modulus of elasticity governs the strength of all the columns.

The tests upon cast iron are of particular value because they were cast from one pouring and afterwards turned down to $\frac{1}{2}$ " diameter in a lathe. These agree with the experiments of Mr Hodgkinson, showing that the skin of the metal has no great effect on the strength of the column.

The cast iron was of splendid quality. It will be seen that Bessemer steel and cast tool steel are always stronger than cast iron.

In testing long columns some elastic deflection was always apparent before the crushing load was reached.

Regarding the Rankine Gordon formula, consider a column of uniform cross section as in fig. 129, and assume a load P has the effect of slightly bending the column. Then the bending moment at the centre is

$$M = P\delta = pA\delta \quad (97)$$

P = load on column.

δ = central deflection.

p = load per unit area (sq. in.).

A = area in sq. in.

than the actual sectional area of the stanchion itself. This is, of course, by reason of the bearing power of the soil or other natural foundation upon which it stands being limited to so many tons or lbs. per square foot according to circumstances, always less than the safe stress intensity of the material of which the stanchion is made. Practical illustrations of this are given in chapters on steelwork. Between the base (usually of masonry, brickwork, or concrete) and the stanchion, in order to distribute the pressure evenly over the base, is interposed a base plate, which in cast-iron pillars is an integral part of the casting, the maximum fibre stress occurring either at the top or bottom of a plate of uniform thickness, or the bottom of a cast-iron plate of varying thickness. The ordinary formulae for base plates are either empirical or based on the theory that this line of fracture of a base plate is a straight line tangent to the exterior of the foot of the column standing on it. The latest formulae at the time of writing are those based on Ricker's experiments, and for a circular plate,

$$t = \sqrt{\frac{3P}{fD}(2122D - L^2)} \quad (103c)$$

We will now consider how shafting behaves when used to transmit power. While this is not intended to be a treatise on mechanical engineering, so much of this work comes under the province of the civil engineer that a few words on shafting will not be out of place, because the stresses set up there have a kind of relation to some of the stresses we have just discussed. For instance, the connecting rod of an engine transmitting the pressure of the steam on to the shafts sets up an enormous bending moment therein. Again, the rods of a large pump and the vertical shaft of a water turbine (unless it is balanced in a manner to be described under water power) are loaded in exactly the same way as a pillar of solid cylindrical section, besides the power they transmit.

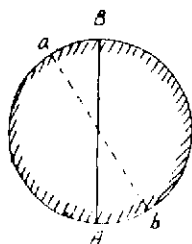


FIG. 131.

Now a man turning a crank such as we described in fig. 99, Chapter VII., if he applied a force of 100 lbs. at the end of a crank 24 in. long, the torque or twisting moment which would be set up in that shaft would be $24 \times 100 = 2400$ in.-lbs. Now, in the same way as a beam having a bending moment, we must have a resisting moment in opposition to it, so we have in a shaft a resistance to torque or twisting moment supplied by the shaft itself. It is called the *Torsional Resistance*, and its value is, in in.-lbs.,

$$T = \frac{F\pi R^3}{2} \quad (104)$$

R = radius of shaft in inches.

F = allowable stress on the material in lbs. (shear stress), or if F = tons, T = inch-tons.

Now the shaft will demonstrate its twist as in fig. 131, and the strain in any ring of fibres is proportional to the arc ab , which is called the *arc of twist*.

Suppose x = any radius in question, and d and D the internal and external diameters of any hollow shaft in question, and f = the allowable shear stress (maximum),

Then the stress in the shaft

$$= \frac{2x}{D} \times f \quad (105)$$

(D this time referring to the diameter of a solid shaft), while for a hollow shaft we have the torsional resistance

$$= \frac{\pi}{16} \left(\frac{D^4}{2} - \frac{d^4}{2} \right) \times f \quad (106)$$

and for solid shafts

$$\frac{\pi D^4}{16} \times f \quad (106a)$$

which compares with Equation 101.

Therefore by comparison we see that for the same length and weight a hollow shaft is 44% stronger than a solid one. Now if we know that in a certain shaft we are transmitting a certain horse-power (= 33,000 ft.-lbs.) at a certain number of revolutions per minute, then we can find the twisting moment in that shaft by the formula

$$TM = 63025 \frac{HP}{N} \quad (107)$$

where N = revs. per minute, and the value of TM is in *inch*-lbs.

Now obviously in a gas or steam engine having a piston of "A" square inches in sec. area and a pressure * (max.) of P lbs. per sq. in., the twisting moment on the shaft if the crank radius is R inches

$$= TM = P \times A \times R \quad (108)$$

But when the shaft is overhung we also have a bending moment in it (similar to a cantilever end-loaded) when l = distance in inches from the centre line of motion to the nearest bearing,

$$BM \text{ (in in.-lbs.)} = P \times A \times l \quad (109)$$

Now the equation for resistance to bending in any solid circular shaft when supported at both ends

$$= \frac{E\pi R^3}{4} \quad (110)$$

that is, just one-half the resistance to torque (*vide* Formula 104), therefore 32 substituted for 16 in Equations 106 and 106a will give us the resistance to bending on those shafts.

When we have a bending moment as obtained by Equation 109 and a torque as in 108 we have to combine the two and get an *equivalent* twisting moment = TM_e and

$$TM_e = BM + \sqrt{BM^2 \times TM^2} \quad (111)$$

In addition to the strength of shafts we also have stiffness to consider, the same as in beams. But in this case the strength varies as D^3 and stiffness as D^4 . Referring to Equation 107, we have a shaft which we know will stand a certain torque of TM in.-lbs., then we can transpose the equation to give us the safe horse power in any shaft, which we can transmit.

Its value is

$$HP = \frac{N \times TM}{63025} \quad (112)$$

Take now the case of a shaft in which we are given the twist in a certain

* Effective pressure = Forward P - Back pressure.

number of degrees = D ; we have the strain also given = x . Then if we want a certain shaft which under the given strain will not have an angle of twist of more than that specified, its diameter, which is found from the circumference by dividing by $\pi = 3.1416$,

$$\text{Circumference of shaft} = \frac{360x}{D} \quad (113)$$

there being 360° in a circle.

But if we want to find the size of a shaft to do a certain specific duty (e.g., the horse-power we want to transmit), find the torque by Equation 107 in in.-lbs. = T .

Then f being = 10,000 for a steel shaft and 7000 for an iron shaft, we have the diameter of our proposed shaft in inches

$$D = \sqrt[3]{\frac{T}{f} \times 5} \quad (114)$$

For instance, required a solid shaft to transmit 1000 h.p. at 100 RPM.

$$\text{Torque} = 63025 \times \frac{1000}{100} = 630250$$

$$D = \sqrt[3]{\frac{630250}{10000} \times 5} = \sqrt[3]{315.125} = \text{nearly } 7 \text{ in.}$$

A 7-in. shaft would no doubt be provided.

Coming again to the angle of twist, and considering stiffness proportional to D^4 , it is obviously measured by the smallness of the angle of twist per unit length of shaft. The simple equation for this is

$$\theta = \frac{2fl}{CD} \quad (115)$$

C being the modulus of rigidity,

D = outer diameter of shaft in inches,

l = length of shaft in inches,

f = applied or allowed skin stress in lbs. per sq. in.,

and θ , much used by engineers, will give us a fraction, say $\frac{1}{15}$, that is, $\frac{1}{15}$ of a radian, which is equal to 57.3 degrees.

$$\therefore \frac{4}{15} \times 57.3 = 15.7 \text{ degrees of twist.}$$

Reverting to Equations 108 and 109, having these values for BM and TM in a shaft, we can find the actual shear stress and tensile stress set up in any shaft. Their values are—

$$\text{Shear} \quad f_s = \frac{TM}{\frac{1}{5} D^3} \quad (116)$$

$$\text{Tensile} \quad f_t = \frac{BM}{\frac{1}{10} D^3} \quad (117)$$

D = diameter of shaft in inches.

CHAPTER X.

ON THE DESIGN OF ENGINEERING STRUCTURES.

THE following pages apply to a great extent to structures which concern the engineer in steel (or iron), but it is not intended that the treatment shall be exhaustive. In the first place it is a wide subject, and is fully treated in some very good text-books at the present day, while, in the second place, the municipal or county engineer does not now usually devote his time to going into designs for large bridges, etc., because there are specialists who devote their whole time to the work, and large firms now usually undertake their own designs as well as erection.

It is, however, quite essential that the engineer under whose supervision the above works will probably be undertaken should have a knowledge of the principles of such designs, because competition is now so keen that firms, in order to put in low tenders, cut the sections of their designs down to the smallest possible safe limit, and sometimes even exceed it. When the engineer has been instructed by his employers or clients to get in designs and tenders for a certain work, he therefore requires to decide which is the best tender. Naturally, a board of unprofessional men often want to take the lowest tender. If, however, they are wise, they consult their engineer first, and hold him responsible for the work. If, therefore, the engineer who looks over the tenders has the requisite knowledge of the design, and selects one as being on reasonable terms, he must carefully go over the drawings, and see whether or not the proposed design is in accordance with theoretical and practical principles. If he finds they are, well and good; if not, he is in a position to deter his employers from accepting such a tender, no matter what the price quoted. Of course, in out of the way places, small structures are usually left in the engineer's hands, and he orders his raw material and supervises the work done by direct labour or otherwise, the local blacksmith being able to do the smaller class of steelwork in a more or less satisfactory manner.

Retaining-walls, earthworks, and foundations—in fact, masonry, brick, and concrete work in general—are left to the engineer entirely, together with his roads, sewerage, and waterworks. These subjects, therefore, will receive a much deeper consideration both from a theoretical and practical standpoint.

A way of analysing the stresses and strains in structures, called graphic statics, by means of diagrams upon the principle of the parallelogram and triangle of forces, is also much used, and is of great value to the engineer. We shall treat this subject in the next chapter.

Design of Suspension Bridges.—Dealing with the design of structures, we must note that—

(1) Ties are always subject to tension, and the equilibrium of a movable tie is stable.

(2) Struts being subject to compression, their equilibrium is unstable?

(3) A brace is a stay bar on which there is permanent stress, and which does not alter the resultant of the forces applied to the joints which it connects, but only the distribution of the components. In fig. 132 we have a hanging chain similar to that in a suspension bridge. The supports are assumed to be at the same level.

$$\text{The distance } x = \frac{a}{2} \quad (118)$$

while m , called the focal distance of the parabola (hanging chains always describe a parabolic curve) [vide fig. 154],

$$= \frac{a^2}{16y} \quad (119)$$

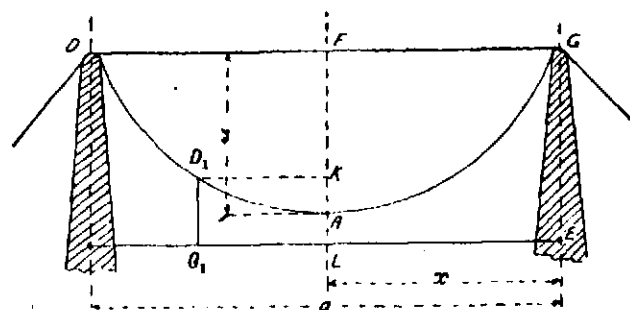


FIG. 132.

The inclinations at the points of support are equal to i ,

$$\tan i = \frac{4y}{a} \quad (120)$$

And if the weight of the chain in lbs. per foot-run = P , the horizontal tension in the chain

$$= H = \frac{Pa^2}{8y} \quad (121)$$

while tension at points of support

$$= V = H \sqrt{\left(1 + \frac{16y^2}{a^2}\right)} \quad (122)$$

the total length of chain

$$= l = m(\tan i \sec i + \text{hyp. log}(\tan i \sec i)) \quad (123)$$

Hyp. log means that the hyperbolic or Napierian logarithms have to be used. If a table is not handy for finding their value, multiply the common log by 2.30258.

But if we had such a case where the supports were not level, the above reasoning would be altered; say, instead of y we had y_1 and y_2 ; then m , instead of the value attached to it in Formula 119, would be

$$m = \frac{a^2}{4y_1 + 4y_2 + 8\sqrt{y_1 y_2}} \quad (124)$$

Such chains when applied to suspension bridges rest upon what is called a saddle (on the top of each pier) of cast iron or steel, upon which they are quite free to slide, and in which case the tensions on each side are equalised. Now if we know the horizontal tension H , Equation 121, we can find the vertical pressure in the piers. Its value is

$$V = 2H \tan i \quad (125)$$

But the inclinations must be equal, and when this is not the case we have i_1 and i_2 , and

$$V = H(\tan i_1 + \tan i_2) \quad (125a)$$

From what we have said we may conclude that the centre of the chain will have the least tension. Say we want to draw the curve similar to that which a suspension bridge chain would describe, we have in fig. 132 the distances FA and AL and also the two piers. Then to find any distance D_1G_1 we have the formula

$$\frac{(AF - AL)D_1 K^2}{LE^2} + AL \quad (126)$$

(N.B. — $LE^2 = x^2$.)

When we have a roadway attached to the suspension chain and find its weight per foot-run, which will usually be somewhere about 5000 lbs., call it Z ; then we can find what strain there will be at the centre of the chain,

$$S_A = \frac{Z}{2(AL - AF)} - 0003 \quad (127)$$

Now if we multiply this strain by .0000893 we have the required sectional area of our chain in square inches. But we said before that the centre of the chain had the least tension. $\therefore$ the strain on the chain varies at different points. So to find the strain at any point, D_1 say (fig. 132),

$$S_D = \sqrt{\left(\frac{2KA}{D_1 K}\right)^2} + 1 \times S_A \quad (128)$$

(S_A found by Equation 127),

and obviously this $\times .0000893$ gives us the sectional area of the chain at the point, and so on. It is usual to consider a factor of safety for suspension bridges of 3 for dead loads and 6 for live ones, and so combine them, as we stated before in Chapter VII. The variation of the load on a suspension-bridge chain, as we have just mentioned, varies as the secant of the inclination of the point on the chain in question to the horizontal. Consequently the weight of such a chain over equal horizontal distances varies as the secant of the inclination likewise. In fig. 132 we see two distances x and y . The usual ratio of these in practice of course varies. Generally it is from 4.5 to 1, to 7.5 to 1.

Again, having found H by Equation 121, we can get two constants = C . Their values are—

$$1. \text{ For wire cables } C = \frac{Hz}{4500} \quad (129)$$

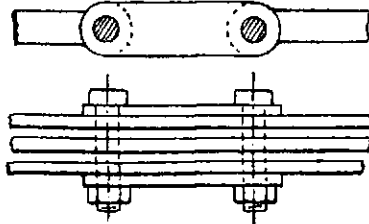
$$2. \text{ For steel link cables } C = \frac{Hz}{3500} \quad (129a)$$

Then the weight of the half-chain of uniform strength is

$$W = C \left(1 + \frac{4y^2}{3c^2} \right) \quad (129b)$$

The method of jointing such chains as case 2 will be seen by reference to figs. 133 and 134, the bars being specially rolled (*not forged*), with enlarged ends, as follows:—

The sum of the width of metal around the bolt should be $\frac{1}{4}$ as much again as the whole bar, the top $\frac{1}{8}$ of the whole, the diameter of the pin being at least $\frac{3}{8}$ the width of the link. They must be tested to at least 15 tons per square inch. If in testing the result shows a permanent elongation of more than $\frac{1}{1000}$, they must be rejected. When two or more chains occur above one another, as in fig. 135, we have to distribute the weight evenly between them. The figure shows how this is effected, the suspension rods usually being 5–12 feet apart. The platforms of suspension bridges nearly always have more or less of a camber. The practice of carrying the chains upon a platform or



Figs. 133 and 134.

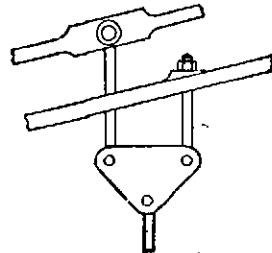


FIG. 135.

truck as described is most effective. The truck, which rests on steel rollers, is usually in length $\frac{1}{10}$ of the span. Hinged piers of cast iron have been used, but the author has no experience of them. In the case of no such arrangements, the chain will set up an overthrowing moment in the piers. If X be any point in the piers at which we require to find the moment, and w_1 the rolling load per foot-run, using the previous notation and having feet and lbs. as our units,

$$X = \frac{w_1 x^2}{2y} \quad (130)$$

When X is multiplied by its distance from the top of the pier, obviously then, on the near side of the pier, we have a compressive stress, and on the far side (away from the bridge) a tensile stress. The load such a pier will have to sustain when

$$\left. \begin{array}{l} W_1 = \text{weight of proposed pier itself} \\ w = \text{dead load of bridge} \\ w_1 = \text{live load of bridge} \end{array} \right\} = W$$

$$P = W + (2w + w_1)X \quad (131)$$

The chain anchorages are termed abutments. A form recommended by Professor Rankine* is shown in fig. 136. The saddle and anchoring plates are shown in the figure. The front ends of such abutments are always made perpendicular

* Rankine's *Civil Engineering*, 23rd ed., p. 576.

dicular to the direction of pull on the chains. In all other points the stability is the same as ordinary bridge abutments (which will receive due consideration) by considering pulls instead of pushes. The saddles in the abutments do not require rollers, but in order to obtain the slight amount of play required by temperature effects they are (or should be) laid on layers of asphalted felt about 6" thick. The tunnels wherein the chains lie should always be accessible for workmen. When suspension bridges have to take a particularly heavy load, such as railway work, for which purpose they have found favour among American engineers, auxiliary girders of the lattice type have to be provided as in fig. 137. They are fastened down at each end, but must in no case be fixed in direction. They are usually made in two halves, connected by pin joints, which have to resist the maximum shear stress. If w_1 as before = the maximum rolling load, this auxiliary girder is subject to a bending moment. Its maximum value is

$$M = \frac{w_1 x^3}{16} \quad (132)$$

while the maximum shear stress set up on the same girder

$$F_s = \frac{w_1 x^2}{4} \quad (133)$$

We shall come to lattice girder design shortly. Such a load has the property of acting alternately up and down, so each part of such a girder must be capable of acting either as a strut or tie. If the girder be not hinged at the centre as described, Equation 132 becomes

$$M = \frac{w_1 x^2}{14} \quad (134)$$

Diagonally braced chains have been designed. The depth, which must be uniform throughout, has a value = $\frac{1}{2}$ the proposed depression of the chain. The object of doing so is that no extra material is then required in its sectional area. It is, however, less effective than the braced girder principle, owing to the joints being very liable to become loose by temperature effects. A system of diagonal bracing between the chain and platform is also in vogue, but such will not be efficient unless they are all subject to tension (*vide* lattice girder design to follow). This, of course, is to ensure that none of the suspension rods shall act as struts, which of course would be ridiculous. The system is also open to the same disadvantages due to temperatures as the braced chain. Before setting about designing such a bridge, in addition to the rolling load, which may be taken on an average of 1 ton per foot-run for every single line of railway, or as the breadth of roadway in feet + 120 (=lbs.) [because this takes into consideration that the greatest load a road bridge will have to bear would be that of a closely packed crowd, and assuming a man to weigh 120 lbs. (a small man), and to cover an area of 1 square foot], we want to find the probable weight of such a bridge.

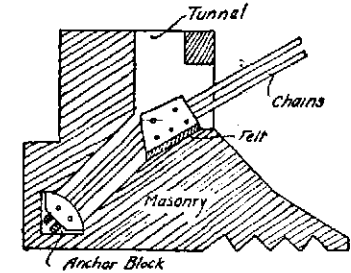


FIG. 136.

We then consult the data of the existing bridges of like construction. Such data are readily obtainable in most libraries and engineering institutions.

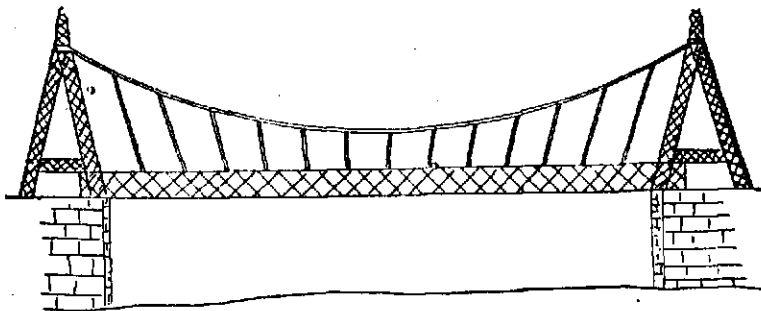


FIG. 137.

Then, calling the span of our proposed bridge in feet l , the gross weight of the superstructure of our existing bridge = W , w_1 proposed live load,

$$L = l \left(1 + 2 \frac{w_1}{W} \right) \quad (135)$$

and $L-l$ will give us the proportion which our proposed bridge bears to an

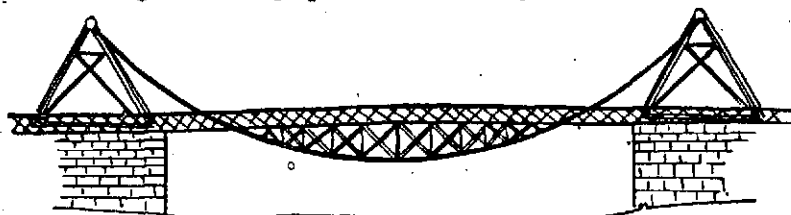


FIG. 138.

existing one. To save trouble, however, we can take as being accurate the following values of L :-

- | | |
|---|-------|
| 1. Tubular bridges, not continuous. | 614 |
| 2. " " continuous | 760 |
| 3. Lattice girder bridges, not continuous | 900 * |
| 4. Ordinary arched ribs | 630 † |
| 5. Suspension bridges | 2000 |

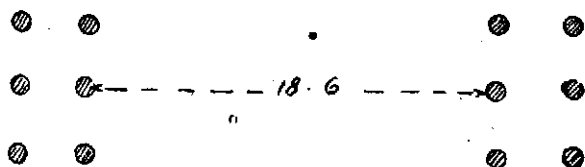


FIG. 139.—Union Bridge, River Tweed.

In figs. 137 and 138 we give examples of typical designs of suspension bridges, and in figs. 139, 140, and 141 the types of chains in three well-known suspension bridges.

* Depth = $\frac{1}{4}$ span.

† Versine = $\frac{1}{4}$ span.

To conclude our remarks upon suspension bridges, let us take an actual example. In fig. 132 (not to scale) $LE=200$ ft., $AF=40$ ft., and $AL=2$ ft.;

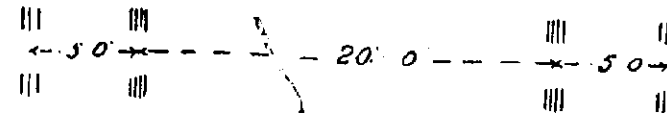


FIG. 140.—Fammersmith Bridge.

weight of roadway when loaded = 5000 lbs. per foot-run. $DK=100$ ft. Then using Equation 126 we have

$$\frac{(40-2)100^2}{200^2} + 2 \cdot 9.5 + 2 = 11.5.$$

$$\therefore D_1 G_1^2 = 11.5.$$

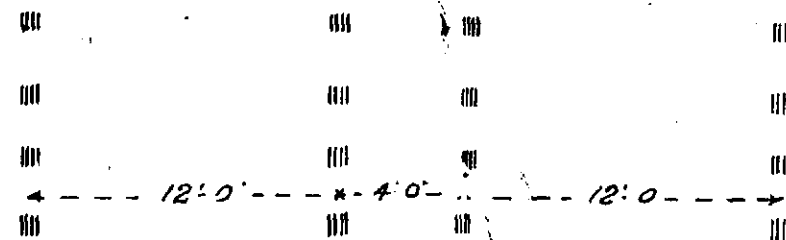


FIG. 141.—Menai Bridge.

Using Equation 127,

$$\left(\frac{2(2-40)}{200^2} - 0.003 \right) = 0.016 \quad \text{and} \quad \frac{5000}{0.016} = 3,125,000 \text{ lbs.}$$

= strain at lowest part of the chain.

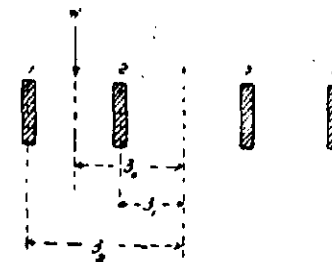


FIG. 142.

Again using Equation 128,

$$S_D = \sqrt{\left(\frac{2 \times 9.5}{100} \right)^2 + 1} = \sqrt{1.0261} = 1.013 \times 3,125,000 =$$

Strain at the point in question = D , this multiplied by $\frac{1}{100}$ gives the correct sectional area of the chain at the point $D=283$ by Equation

In the same way all the other points can be calculated.

Girder Bridges.—Take a bridge to take a double line of girders. The usual arrangement of girders will be similar to fig. 142. Each of

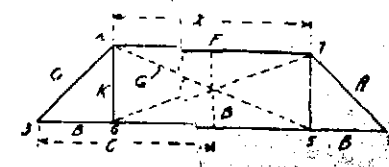


FIG. 143.

2. *The Booms.*—We are supposing our bridge is for an ordinary roadway, and are taking the load per foot-run as 2 tons = $2 \times 60 = 120$ tons = $\frac{120}{8} = 15$ tons dead load on each joint = w . We are also providing for a live load of 5 tons in each joint = w_1 .

Then by Equation 141 we find the stress on the boom at the centre

$$= \frac{(15+5) \times 60}{10} \times \frac{4(8-4)}{2 \times 8} = \frac{20 \times 60}{10} \times 1 = 120 \text{ tons.}$$

In the same way we can find the others, and have our values so:—stress on the booms =

At the centre	120 tons
„ joints 3 and 5	114 „
„ „ 2 „ 6	90 „
„ „ 1 „ 7	64 „ or nearly $\frac{1}{2}$ that at the centre.

Assuming our metal to stand 5 tons per square inch in tension or compression, we have for the main booms, say an I beam, $16'' \times 6'' \times 62$ lbs., having a total sectional area of 18.23 sq. in. Then to take the extra load between 3 and 5 we add a $6'' \times \frac{1}{2}''$ plate, and between 2 and 6 another $6'' \times \frac{1}{2}''$ plate likewise, to provide for stresses of 120 and 114 tons.

Coming now to the diagonal bracing, we use Equation 144.

We have for the two diagonals at 3 and 5 a load of

$$\begin{aligned} & \left(\frac{15 \times 12.5}{10} \times \frac{8+1}{2} - 3 \right) + \left(\frac{5 \times 12.5}{10} \times \frac{(8-3)(8-3+1)}{2 \times 8} \right) \\ &= (18.75 \times 1.5) + \left(6.25 \times \frac{30}{16} \right) = 28.125 + \frac{93.75}{8} \\ &= 28.125 + 11.72 = 39.845, \text{ say } 40 \text{ tons.} \end{aligned}$$

In a similar way we get the other calculation, and have

For two diagonals at 4	18 $\frac{1}{2}$ tons.
„ diagonals at 3 and 5	40 „ (above)
„ „ 2 „ 6	68 „
„ „ 1 „ 7	91 „

Say at 4 we use two flat bars $3\frac{1}{2}'' \times \frac{1}{2}''$, kept apart by suitable distance pieces.

At 4 we provide	$3\frac{1}{2}'' \times \frac{1}{2}'' \times 5.95$ lbs. flat bars.
„ 3 and 5 we provide	$6 \times 3 \times 14.5$ lbs. channel.
„ 2 „ 6 „	$9 \text{ in.} \times 3\frac{1}{2}'' \times 25.4$ lbs. channel.
„ 1 „ 7 „	$12 \times 3\frac{1}{2}'' \times 32.88$ „ „

Vertical Rods.—By Equation 142, at 3 and 5 we have

$$\begin{aligned} & 15 \left(\frac{8+1}{2} - 3 \right) + 5 \left(\frac{(8-3)(8-3+1)}{2 \times 8} \right) \\ &= 15 \times 1.5 + 5 \times \frac{30}{16} = 22.5 + \frac{75}{8} = 22.5 + 9.37 = 32.0 \text{ approximately.} \end{aligned}$$

in the vertical rods the stresses on the members are

at 3 and 5	32 tons.
„ 2 „ 6	51 „
„ 1 „ 7	70 „
„ centre (using Equation 143)	80 „

So to take up these stresses we provide

at 3 and 5	6 in. $\times \frac{1}{2}'' \times 10.20$ flat bars.
„ 2 „ 6	$7 \times 3 \times 17.5$ channel.
„ 1 „ 7	$9 \times 3.5 \times 25.40$ channel.
„ centre	$10 \times 3.5 \times 28.21$ „

Now let us work out Equation 145. We have

$$\begin{aligned} & - \frac{15 \times 12.5(8+1)}{10} \left(\frac{2}{2} - 3 \right) + \frac{5 \times 12.5}{10} \times \frac{3(3+1)}{2 \times 8} \\ &= -18.75 \times 1.5 + 6.25 \times .75 \\ &= -28.125 + 4.68, \end{aligned}$$

a negative result. And when Formula 145 gives us a negative result, we know that our live load is *not* large enough to warrant us putting on the thin line diagonals, assuming wind pressure at 40 lbs. per sq. ft. applied to one-half the surface of the girder = $60 \times 40 \times 5 = 12,000$ lbs. $\div 7 = 1700$ lbs. at each joint, which is much less than requires a square inch of section, but we will use $4' \times 1\frac{1}{4}'' \times 5''$ lbs. angles at each joint, placed between the main booms on top. They may be curved to give extra headway, for the flooring we shall use $60 \times 25 = 1500$ sq. ft. of troughing. We will have ample strength in the size known as "D. min." if used over the span of 26 feet = width between booms. Its weight per sq. ft. is 28.75 lb. $\therefore$ exclusive of the concrete it would be 1500×28.75 lbs. Then summing up the quantities obtained we have

	Lbs.
Booms	6,510
Diagonals	918.00
Vertical Rods	148.75
Wind bracing, $180' \times 5$ lbs	362.5
Flooring, 1560×28.75 „	635.0
Distance pieces 100 lbs.	822.0
Rivets (snap heads) 225 lbs.	204.0
Contingencies 200 lbs.	350.0
	508.0
	282.0
	940.0
	44,850.0
	100
	225
	200
Total	57,054
Total excluding flooring	12,204

$\therefore$ total weight of bridge (steelwork only)

$$\begin{aligned} &= 2 \times 12,204 + 44,850 \\ &= 24,408 + 44,850 = 69,258 \text{ lbs.} \end{aligned}$$

$\div 2240 = 31$ tons (nearly).

under the rails. When any pairs are loaded we have certain stresses in the others. We want to find the value of such stresses. Using the lettering in the figure, the first thing is to find a coefficient

$$= c = \frac{wz}{2(z_1^2 + z_2^2)} \quad (136)$$

Then the load on the other girders is as follows:—

$$\left. \begin{aligned} \text{Load on 1} &= \frac{w}{4} \cdot z_1 \quad (a) \\ \text{" 2} &= \frac{w}{4} \cdot z_2 \quad (b) \\ \text{" 3} &= \frac{w}{4} \cdot z_3 \quad (c) \\ \text{" 4} &= \frac{w}{4} \cdot z_4 \quad (d) \end{aligned} \right\} \quad (137)$$

The load on No. 4 will often prove negative, which shows that the member is pulled upwards by the load. Fig. 143 shows what we call a trapezoidal

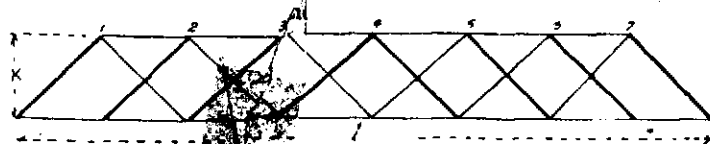


Fig. 144.—Lattice Girder.

truss. We want to find the BM in the horizontal members. It will be greatest when the load is at 4 or 1. Its value is

$$M = \frac{wx(c-x)}{2c} \quad (138)$$

Again, we have stresses in the diagonals. These are acting as struts, the verticals acting as ties, while the top member is the straining piece. The value of the stresses on the diagonals

$$= \frac{w}{2cK} \left(\frac{c-x}{2} \right) \sqrt{4x^2 + K^2} \quad (139)$$

Fig. 145 shows a lattice girder. If the load were dead only, the diagonal members shown by thin lines would not be required. The verticals are tension rods. First of all we want to find the bending moment at any point (vide fig. 145).

N = number of divisions in the bottom chord, and n = the joint at which we require the BM denoted by its number, we have

$$M = \frac{n(N-n)}{2N} wl \quad (139a)$$

So, having our line diagram, we have the following formulæ to give the stresses in each of the members:—

H = thrust in upper boom and tension in lower one.

V = tension in vertical rods.

V_1 = " on central vertical rod.

T = " or compression on large diagonals.

t = " " " small " "

K = height of girder } feet.

l = span " " }

S = length of diagonals, feet.

$$\text{Then } S = \sqrt{K^2 + \frac{l^2}{N^2}} \quad (140)$$

by principle of sum of squares.

$$H = \frac{(w + w_1)l}{K} \cdot \frac{n(N-n)}{2N} \quad (141)$$

$$V = w \left(\frac{N+1}{2} - n \right) + w_1 \cdot \frac{(N-n)(N-n+1)}{2N} \quad (142)$$

$$V_1 = w + \frac{w_1}{4} \cdot \left(\frac{N}{2} + 1 \right) \quad (143)$$

$$T = \frac{wS}{K} \left(\frac{N+1}{2} - n \right) + \frac{w_1S}{K} \cdot \frac{(N-n)(N-n+1)}{2N} \quad (144)$$

$$t = -\frac{wS}{K} \left(\frac{N+1}{2} - n \right) + \frac{w_1S}{K} \cdot \frac{n \cdot (n+1)}{2N} \quad (145)$$

w = load per foot-run $\times$ distance between each joint = load on each joint (dead). w_1 = greatest live load on each joint = 2 tons per foot-run on an average.

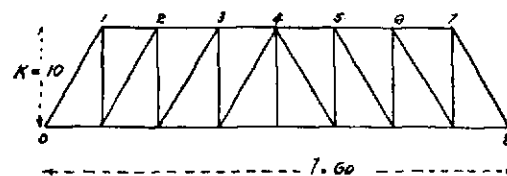


Fig. 145.

Thus we can find the actual load in each member, and thereby obtain suitable scantlings, allowing good steel a safe load of 6.5 tons per square inch.

We will take a practical example to illustrate our reasoning on lattice girders.

Say the span was 60 feet divided into 8 divisions, as shown in fig. 145. The depth of proposed girder is 10 feet.

1. The first thing to do is to find the length of the diagonals by Equation 140.

$$\begin{aligned} &= \sqrt{100 + \frac{600}{8 \times 8} = \frac{225}{4} + 100 = \sqrt{100 + 56}} \\ &= \sqrt{156} = 12.49 \text{ ft., say } 12.5 \text{ ft.} \end{aligned}$$

struts having their ends fixed, we have the formula for ultimate load per square inch of section

$$= \frac{36000}{1 + \left(\frac{l^2}{36000r^2} \right)} \quad (153)$$

l = length in inches and r = radius of gyration, also in inches, but if hinged ends put 3600 in the divisor, a factor of safety of 5-6 being used, allowing a safe load of 12,500 lbs. per sq. in. of section. When joints occur in sections of cast iron such as we referred to in the arched ribs, the two abutting flanges should always be faced in the lathe,* and the total area of bolts at least equal to that of the top flange. When I beams give way under a transverse load they do so by lateral bending of the top flange; but having regard to the shear stress in a beam, as shown in fig. 148, we have the formula for it equal to

$$S = \frac{36000}{1 + \left(\frac{s^2}{30000t^2} \right)} \quad (154)$$

t being the plate thickness in inches, and s = distance along a line drawn



FIG. 148.

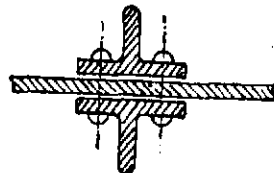


FIG. 149.

at 45° from the point in question to the web-stiffeners, or if such are absent, to the ends of the beam, a factor of safety of 6 being employed. Then by allowing 6 tons per sq. in. of section, we can find the area of the shear members. The web-stiffeners are usually two tees, as shown in fig. 149, in which case the maximum live load on them is treated as in Formula 153. It is due to the reasoning in fig. 148 that we have the diagonal members of a warren girder. We gave before a formula for deflection of a beam of uniform section. In a built-up girder of the warren type the working deflection must not be greater than

$$v_1 = \frac{l^2}{10000K^2} \quad (155)$$

l and K being the same notation as before in inches, then v_1 will be in inches also. On p. 123 we noticed a form of troughing used as a floor in our bridge design. For small foot-bridges, etc., this would be rather heavy. We shall describe it, of course, later, but in light bridges it is often replaced by corrugated iron, or what is known as Mallet's buckled plates. For the corrugated iron, if we have

a = breadth of the plate (or rather sheet)
 r = depth from ridge to furrow
 t = thickness of plate

c = modulus of rupture of the material = 34,000 lbs. per sq. in.
 the breaking load on such a plate,

$$= \frac{4}{15} c. a. r. t. \quad (156)$$

* Or slotting machine.

which is easily remembered. But in Mallet's plates the dimensions are shown in fig. 150. They must be fixed all round, and are then twice as strong as when only two sides are fixed, but when using Equation 156 we take $c = 21,000$ instead of 34,000 as in the case of the corrugated iron. Now the truss which was illustrated and described in fig. 143 is often used inverted, but in either case the following equations are applicable, the pulls becoming pushes (and vice versa) in some of the members. The tension in each of the ties A and G

$$= T = \sqrt{H^2 + W^2} \quad (156a)$$

The thrust on BB also = the tension on A is equal to

$$= \frac{W(c-x)}{K} \quad (157)$$

where W = maximum load on any point + $\frac{1}{4}$ the actual weight of the truss. But such a truss is always subject to unequal loading, so denoting the excess

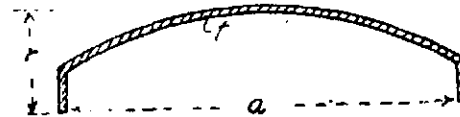


FIG. 150. — Mallet's Plate.

of weight on either 5 or 6 by W_1 we can then find the maximum tension on G the diagonal (dotted) braces and K = depth of truss,

$$= S = \left(\frac{G_1}{K} + W_1 \frac{C-x}{2cK} \right) \times \sqrt{4x^2 + K^2} \quad (158)$$

G_1 being the actual weight of each of the braces. The greatest thrust on K is

$$= V = W_1 \times \left(\frac{C-x}{2c} \right) + \frac{B}{4} + K + G_1 \quad (159)$$

In warren girders the maximum bending moment occurs when the load is evenly distributed, and the maximum shear when a large part is loaded and a small part unloaded. Hence we always calculate upon the former when designing the booms and upon the latter when designing the shear members, that is, the diagonal bracing. The depth is best made about $\frac{1}{4}$ the span. Assuming we have made the usual calculations in reference to a proposed warren girder, it will be well to check over the sections obtained thereby by the following formula:—

P = the allowable safe stress in each member

$$P = 1 + \left(\frac{8000}{40000r^2} \right) \quad (160)$$

the notation being the same as usual in this now familiar formula to the reader. This, when applied to compression members, assumes square abutting ends. When we have 1 pin and 1 square end, substitute 30,000 for 40,000 in the divisor, and if two pins or a tension rod, substitute 20,000. It is owing to the failure of cast iron when a certain ratio of l to d (usually 30) is exceeded that such material is not now much used for members of steel bridges, even in compression.

Most railway bridges are designed for a rolling load of 1 ton per foot-run. When of lattice work the joints between the lower boom and the braces are often large cylindrical pins, while in bowstring girders the bow usually springs from the socket, and its form should approach the parabolic in theory, but is usually made circular. The tie is made fast by a jib and cottar arrangement (fig. 147).

Take a bowstring girder having a span of l ft. with a maximum rise $= K$, w = dead load per foot-run and w_1 = rolling load likewise. Then we have the value of the greatest horizontal thrust and the boom

$$= H_c = \frac{(w_1 + w)l^2}{8K} \quad (161)$$

while at any point x feet from the centre

$$H_x = \sqrt{\{H^2 + (w + w_1)^2 x^2\}} \quad (162)$$

(x at the springing being of course $\frac{l}{2}$),

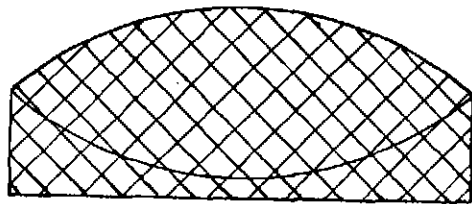


FIG. 151.—Saltash Bridge.

while on the diagonals the greatest tension is equal to

$$H_d = \frac{(w + w_1)l}{N} \quad (163)$$

N being the number of divisions. Some of the uprights may, however, act as struts. And to find out if such is the case we have the formula

$$\frac{l}{N} \left(w \times \frac{n(n+1)}{2N} - w_1 \right) \quad (164)$$

n and N being the usual notation. This occurs under partial load. If such turns out positive we have a thrust, and if negative, tension. If it is nil we may expect a very small tension resulting therefrom. In the latter case, of course, we may do without these diagonals, and simply treat the bridge hinged at the ends, by the reasoning which we have demonstrated earlier in the chapter. There is no actual need for the tie to be horizontal either; it may easily take the form of a catenary, in which case we make the thrust due to the weight of the rib = tension due to the weight of the tie. It was on this principle that the well-known Saltash bridge built by Brunel for the G. W. Railway Co. is designed. Its form is that shown by fig. 151. When we have arched ribs as before suggested, and of which we worked out an example, we want, of course, spandril bracing, fig. 152, in which case we have H_c = maximum horizontal thrust at the centre,

$$= H_c = \frac{Wc^2}{2K} \quad (165)$$

c being the $\frac{1}{2}$ span and K the rise, and $W = w + w_1$. Again, if we want to find the sectional area at any point at an inclination i with the horizontal,

$$A = \frac{H_c \sec i}{f} \quad (166)$$

f for steel = 6 tons per sq. in.

Regarding wind pressure on bridges, we assume the *maximum* in Britain equal to $2\frac{1}{2}$ tons on every 100 sq. ft. of exposed surface.

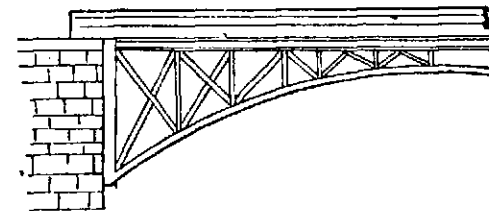


FIG. 152.—Spandril Bracing.

To calculate the exposed surfaces we proceed as follows:—

1. Height of train $\times$ length of girder, plus—
2. Actual surface of girder above and below train and in braced girders, the whole of the girder when the open spaces in the windward girder are more than $\frac{2}{3}$ of the total area. $\frac{2}{3}$ when the open spaces are between $\frac{1}{3}$ and $\frac{2}{3}$ of the total area, and $\frac{1}{3}$ when open spaces are less than $\frac{1}{3}$ the total area.

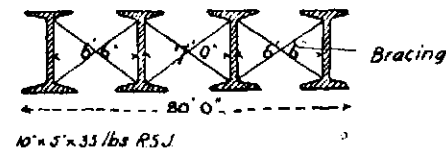


FIG. 153.

The following paper by Professor Genese, of University College, Wales, is well worth the perusal of the reader.*

His views were as follows:—

The Standardisation of Formulæ in Engineering.

"In the first place, he thought it only right to say that, in his opinion, teachers and authors dealing with purely mathematical subjects might be absolved from blame in respect of the existing confusion in engineering notation. The symbols and signs employed in algebra, arithmetic, geometry, and trigonometry stood by themselves, and caused no confusion even when applied to mathematical demonstrations, frequently necessary in connection with engineering problems.

"Similarly, the symbols employed in various branches of physical science were not necessarily to be regarded as conflicting with the symbols used in engineering formulæ. Nevertheless, it was the fact that when the domain of applied physics had been entered, various symbols became necessary for the purpose of denoting quantities with were common to various branches of that science, and to what might be distinguished as engineering practice.

"The symbols which teachers of physical science applied in a general sense

* *The Architect.*

A further example of the difficulties innocently caused by those who published formulæ based upon divergent systems of notation was furnished by the following small selection from the numerous equations for the moment of resistance of reinforced concrete beams, the moment being taken in each case about the centre of tension in the reinforcement.

The equations are:—

R.I.B.A. Committee

$$M = \frac{1}{2} A_c c (d - \frac{1}{3} k d) \quad (1)$$

Marsh

$$M = \frac{c u b}{2} \left(h - \frac{u}{3} \right) \quad (2)$$

Johnson

$$M_0 = \frac{P_c}{3} (3 y_2 + 2 y_1) \quad (3)$$

Sabin

$$M_0 = z f_c \frac{y_1}{2} \left(\frac{2 y_1}{3} + y_2 \right) \quad (4)$$

Thompson

$$M_a = \frac{C x b d^2}{2} \left(1 - \frac{x}{3} \right) \quad (5)$$

Notwithstanding their apparent dissimilarity, all these formulæ were based upon the same theory; and if rewritten with standard symbols, they reduced to absolutely identical terms.

Thus, using the notation of formula (5), equations (1) to (4) became:—

$$\left. \begin{aligned} M &= \frac{1}{2} b d x c \left(d - \frac{1}{3} d x \right) \\ M &= \frac{C d x b}{2} \left(d - \frac{d x}{3} \right) \\ M_0 &= \frac{1}{3} C b d x \left[3(d - d x) + 2 d x \right] \\ M_a &= b C \frac{d x}{2} \left[\frac{2 d x}{3} + (- d x) \right] \end{aligned} \right\} = \frac{C x b d^2}{2} \left(1 - \frac{x}{3} \right)$$

While the discrepancies between the equations in their original form were distinctly calculated to cause perplexity and to create distrust, the standardisation of the same equations obviated all uncertainty and imparted confidence in their reliability by furnishing unmistakable proof that all the equations were based upon the self-same theory.

CHAPTER XI.

ARCHES.

The Catenary is the form in which a uniformly loaded chain hangs when supported at its ends, and all such catenaries are similar. In fig. 154, AO is the "parameter" or modulus of the catenary = m , and on this quantity all its dimensions and other quantities will depend. Let B be another point in the curve, call OX x , and BX y . Then the length of the arc

$$AB = S = \sqrt{y^2 - m^2} \quad (167)$$

and by making XT = m and XT B = a right angle, TB = AB = $S = \sqrt{y^2 - m^2}$, as above Formula 167. Now to find area of AOXB we have the formula

$$A = \frac{m^2}{2} \left(\log \epsilon \frac{x}{m} - \log \epsilon - \frac{x}{m} \right) \quad (168)$$

in the case of the true catenary. Now call the weight of the chain per foot-run p , then the horizontal tension at A

$$= p m \quad (169)$$

It may here be remarked that $\log \epsilon$ signifies that the hyperbolic or Napierian logarithm of the quantity $\frac{x}{m}$ is to be taken. Tables of such logs appear in most pocket-books of engineering formulae, or if a table of common logs only is handy, we may multiply such a common log by 2.30258, and so attain the desired quantity. Now the vertical load between AB is obviously pS , and the tension at the point B = py , etc. In practical applications, however, the parabola is used in preference to the true catenary, being simpler of calculation, so we have in the case of a parabola

$$y = m \left(1 + \frac{x^2}{2m^2} \right) \quad (170)$$

But we can have what is known as a transformed catenary with the same horizontal ordinates but different vertical ones, but they must be altered in the same ratio, viz. (fig. 154) if $Aa = \frac{1}{2} AO$, Bb must equal $\frac{1}{2} BX$, etc. Then in such a case the same reasoning as the common catenary here holds good, viz. the load at B is proportional to XB, and ab is :: O.a. b.X. All the horizontal ordinates being the same as the original ones, the vertical ones being altered in the ratio Aa to AO. By turning any catenary upside down we have the linear or equilibrated arch which forms the line of resistance of real arches when they are in equilibrium

and with much freedom to quantities of this kind were not always the same as those which engineers applied in a specific manner and, with some restrictions, to quantities of a similar nature. Hence the elements of confusion at once became evident.

"They could not reasonably expect science teachers to use one code of notation for mathematical theories and another code for practical applications of the same theories, and it was only natural that professional scientists who wrote engineering treatises should import some of the symbols which they had been accustomed to employ elsewhere.

"A familiar example of importation in this way was furnished by the use of letters such as P, Q, R, and S, or p, q, r, and s, to denote forces or stresses for which engineers had already provided other and more easily understood symbols.

"Unfortunately, however, engineers had not yet established by custom or otherwise a clearly defined system of notation, and until they did so there was no just reason for imputing blame to scientists for introducing into engineering literature any symbols which seemed inconvenient.

"The first thing that they as engineers ought to do was to formulate a code of notation suitable for all the factors in general use, and then to confer with those engaged in the pursuit of the more mathematical branches of science with the object of ascertaining whether a common code of notation could be settled for such factors.

"If such a result should ultimately be found impracticable, the notation applying to purely mathematical theories involved in engineering practice could easily be left to stand on the same basis as that necessarily occupied by the notation employed in algebra and geometry.

"Thus it became evident that the scope of the general question raised by the discussion might turn out to be far less extensive than at first sight appeared probable, although he fully admitted the difficulties by which it was surrounded. He could not help thinking that the greatest obstacle to standard notation was the entire absence of any organisation for the interchange of views between those who were primarily responsible for the publication of engineering formulæ, and who, so long as they were obliged to act independently, could not well avoid the perpetuation and multiplication of conflicting symbols.

"The confusing state of engineering notation was illustrated by haphazard reference to half a dozen treatises by different writers on any branch of engineering work.

"For example, in the department of structural engineering nearly all the factors connected with the design of beams and columns were generously provided with at least three alternative symbols, and some of them had from six to a dozen if they took into account symbols with suffixes.

"This state of things was sufficiently perplexing in itself, and was rendered even more so by the fact that symbols applied to certain factors by one writer were applied by other writers—and sometimes by the same writer—to totally different factors.

"Thus the student of formulæ found that *total loads* or *concentrated loads* were denoted by W, P, and F, and that P and F were also employed to represent *Tension*, *Compression*, *Shear*, and *Reactions*.

"*Shearing force* was indicated by the letters F, Q, and S; Q also stood for the *reaction* of beam supports, and S for the *section modulus* and the *sectional area* of beams, the latter being a factor which was also denoted by a or A.

"Bending moment was represented by M and μ , these symbols at the same time standing for *moment of resistance*, the latter being distinguished in some books by the symbol R. In one book the letter R stood for what was there termed the *moment of resistance*, but the quantity actually intended was the *section modulus*, for which other authorities provided the alternative symbols Z and S."

Authors were unable to agree even upon so simple a matter as the proper symbol for the depth of a beam, some using *d*, others *h*, and others again *H*, this last symbol being sometimes employed for the totally different purpose of indicating *horizontal flange stress*.

The *sectional area* of a beam was *a*, *A*, or *S*, according to fancy; the *deflection* of a beam was endowed with a comprehensive vocabulary, including *d*, *D*, *u*, *v*, *y*, δ , and Δ . The *deformation* of structural material was denoted by *e*, *x*, *a*, β , and λ , and the *radius of gyration* was alternatively expressed by the letters *i*, *h*, *r* and κ (*kappa*).

The category might easily be extended to great length, but the few examples given were quite sufficient to illustrate the chaotic state of things which existed.

The desirability of standardisation was further emphasised by the subjoined forms in which Gordon's and Rankine's well-known column formulæ are presented in some standard treatises and books of reference.

Author.	Gordon's Formula.	Rankine's Formula.
Rankine	$\frac{P}{S} = f \div 1 + \frac{a l^2}{j^2}$	$\frac{P}{S} = f \div 1 + \frac{f^2}{c r^2}$
Ewing	$P = \frac{f}{1 + a \left(\frac{l}{d} \right)^2}$	$p = \frac{f}{1 + c \frac{l^2}{k^2}}$
Goodman	$P = \frac{S}{1 + a \frac{l^2}{d^2}}$	...
Anglin	$P = \frac{f S}{1 + a r^2}$	$P = \frac{f S}{1 + c l^2}$
Molesworth	$P = \frac{F}{1 + k H^2}$	$P = \frac{F}{1 + k l l^2}$
Rivington	$R_c = \frac{A r_c}{1 + a l^2}$	...
Atherton	$P = \frac{f A}{1 + c r^2}$	...

The most noteworthy of these versions are those of Molesworth, which appeared to suggest the absence of any difference between the formulæ of Gordon and Rankine, but the others were fairly distracting in respect of notation, apart from the variations of algebraical form which they exhibited.

All the above calculations can then be used by substituting pushes for pulls. In linear arches the intrados is the actual line of that arch, the

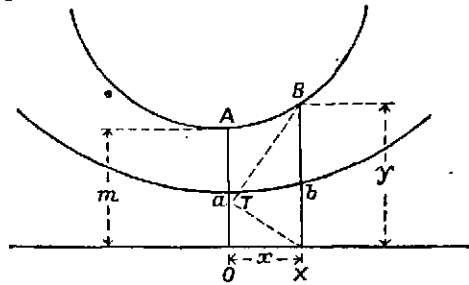


FIG. 154.

extrados being a line above joining the ordinates, and of a length proportional to the load. We therefore see that the extrados of a catenary or transformed catenary is a horizontal line. Now the thrust round any circular ring under a uniform press = pressure per sq. foot of circumference $\times$ radius in feet and denoted by pr (171)

but from such a circle we can project an ellipse on which the vertical ordinates remain the same, but the horizontal ones increase (or decrease) in a certain ratio, c . Then, as above, pr still holds good for the vertical pressures, but the horizontal ones now become cpr (172) the total thrusts being as the semidiameters or axes to which they are parallel. But the intensities of the pressures are as the axes squared, or, conversely, the ratio of the axes of the rib are as $\sqrt{\text{of the ratio of intensities of principal pressures}}$; hence c , as mentioned above in Formula 172, is equal to

$$c = \sqrt{\frac{P_y}{P_x}} \quad (173)$$

P_x and P_y being the intensities of the vertical and horizontal pressures respectively. In any linear arch the radius must be inversely as the pressure. The same reasoning applies to what is known as the hydrostatic arch, which is a form of rib for sustaining a normal pressure such as that of water. Its form is shown in fig. 155. The construction is such that each

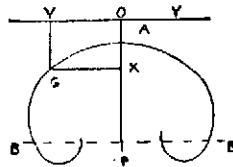


FIG. 155.



FIG. 156.

radius is inversely proportional to the mean vertical distance of its centre below Y, Y, the portion above B.A.B being alone suitable for supporting loads. A spring bent as shown in fig. 156 forms of itself a hydrostatic arch, provided that when free, that spring lies straight. A linear arch suited to sustain earth pressure is known as a transformed hydrostatic arch or geostatic arch, the vertical pressure P_x being proportional to the depth below the surface, and the horizontal pressure P_y bearing a ratio to x of c , depending on the backing. So, having drawn any hydrostatic arch (fig. 155), the geostatic arch will have the same vertical ordinates, but the horizontal ordinates will be proportional to

$$c \text{ and } c = \sqrt{\frac{P_y}{P_x}} \quad (174)$$

the same as in Formula 173, which bears out the above reasoning about pressures and axes, the radius of curvature at the springing being altered in the ratio $\frac{1}{c}$ and at the crown as c^2 to 1. Then if the total loads on $\frac{1}{2}$ any hydrostatic arch be P , those loads on a geostatic arch become

$$P_1 = P c \cos i \quad (175)$$

i being the angle of inclination to the horizontal of the conjugate pressures. However, these pressures are usually horizontal, in which case $\cos i = 1$. Now take any linear or other arch AG, fig. 157, and draw oc (fig. 157A) parallel to a tangent at C and set ob vertical, make $oh = \text{load on } ac$ ($= pS$ as stated before), S being found by Formula 167, then hc being drawn horizontal to cut oc gives us the horizontal component of the thrust along the rib at C. This construction, however, fails at the crown, and we resort to the formula (as in Formula 171)

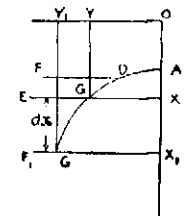


FIG. 157.

$$T = pr \quad (175a)$$

In the same way we draw $oh_1h_1c_1$ and any convenient number of such points so found. Then $hc - h_1c_1$ gives us the thrust on GE, F.G. It will thus be seen that H_1 is less than H . Therefore if this arch is to be in equilibrium, pressure is required through those layers in which H increases in going downwards. Such a pressure may be either a tension from without or a pressure from within. Now $Kd = \text{maximum horizontal thrust}$. Now such a point D (fig. 157) where the thrust on the rib is parallel to oc (fig. 157A) is the point of max. horizontal thrust; such a point is known as the point of rupture of the arch, and the angle doa (i_0) = angle of rupture. The above reasoning leads us to infer that the catenary is one of those few curves in which the length of the arc between any two points can be expressed in terms of the position of those points (Formula 167), and the area bounded by such an arc expressed by a simple formula; also if a horizontal line be drawn at any particular distance below the lowest point of the catenary, the height of any point on the curve above that line = the tension in the hanging chain at the point chosen. That particular distance is found by a formula which is too complex to give here.

This horizontal line is known as the directrix. Another important formula in reference to arches is that to find the conjugate pressure P_y (referred to in Formula 173) exerted in any horizontal layer. Referring to fig. 157A, making $hc - h_1c_1 = H$, then

$$P_y = \pm \frac{H}{d.c.} \quad (176)$$

and it will be negative under conditions explained when referring to tension from without or pressure from within being required.

Practical Design of Arches.—We now come to the more practical side of arch design. The centre of resistance of every arch must be in the middle

angles to the radius at that point. Then, in reference to fig. 160, the following reasoning will hold good, viz. :—

$$RM : KL :: PQ^3 : PB^3 \quad (179)$$

PB being parallel to a tangent at R, and PQ to a tangent at K. It will be seen that the arch in question will not be in equilibrium. Now for a circular

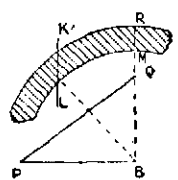


FIG. 160.

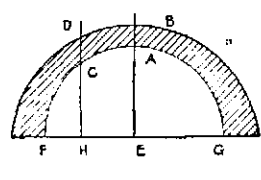


FIG. 161.

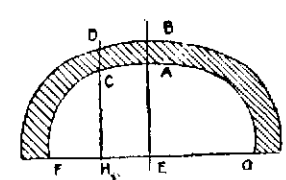


FIG. 162.

arch the conditions of equilibrium are as in fig. 161, when, having determined BA by Formula 178, we can make any depth DC

$$= DC = BA \times \frac{AE^3}{CH^3} \quad (180);$$

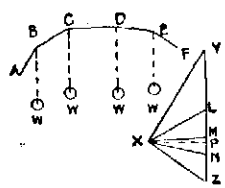


FIG. 163.

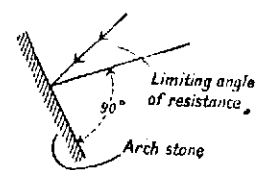


FIG. 163A.

But if the arch is elliptical, as in fig. 162, DC is equal to

$$BA \times \frac{AE^3}{CH^3} \quad (181)$$

But in this case $AE = \frac{1}{2}$ the minor axis and $CH =$ the vertical depth from



FIG. 164.

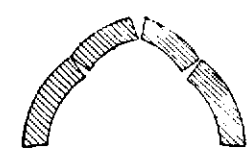


FIG. 165.

C to a horizontal line passing through the centre of the major axis. When such conditions are carried out the arch is in equilibrium.

Arches may fail by the sliding of the voussoirs (or arch stones), which is not a likely occurrence, unless the line of pressure makes an angle equal to or greater than the limiting angle of resistance with a perpendicular to the joint, as in fig. 163A; and as the limiting angle of resistance in such work is usually 30°, we are not likely to see such failure. But, on the other hand, an arch may quite easily fail by turning on some of the joints, as in figs. 164 and 165. When

the line of pressure lies outside the arch as in fig. 164, the inner joints tend to open; if, on the other hand, it lies inside the arch, the outer joints tend to open. Hence the very necessary use in arch design of Formulae 179-180-181. Such a failure as in fig. 164 would be due to too great a load, while that in fig. 165 is due to insufficient depth (see Formula 178) at the crown.

Due to conditions below stated, an arch has the same pressure tending to crush the keystone as it has on its abutments, and the pressure = load per square foot on the keystone $\times$ radius in feet at that keystone. In iron or timber arched ribs we have only two members to consider, viz. the two halves of the arch, which may be considered as two huge voussoirs. So having made a sketch design of the proposed rib DEAB, fig. 165A, and round its centre of gravity = G (which process is a somewhat complex affair, and the reader is referred to any of the standard works on mensuration), draw FH perpendicular to and through the centre of ED, and similarly FI to BC; then if these two lines intersect in FM, which is a vertical line through G, the arch is safe; and if FM refers to the weight of half the rib, FI and FK to scale represent the two thrusts. Then, as x is to y , so

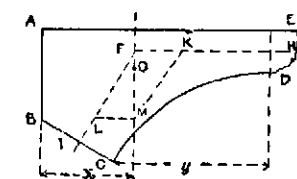


FIG. 165A.

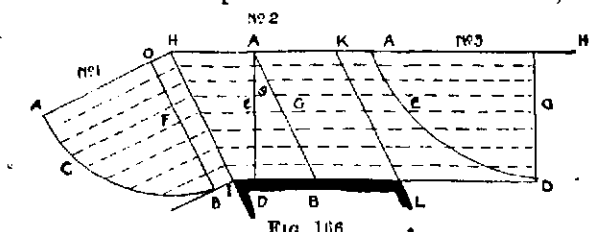


FIG. 166.

are the horizontal thrusts at H and I to the weight of half the rib. We shall treat the subject of arched ribs of timber and iron more fully in a later article under Bridges of Iron and Steel; we may also refer to the previous chapter.

Skew Arches. - An arch which has its centre line not perpendicular to the face of that arch is termed a "skew" arch. It is obvious that the span of a skew arch on the face is greater than that on the square (viz. perpendicular to its centre line) in proportion to the sine of the angle of skew, or, in other words, if the span on the face be multiplied by the sine of the angle of skew, we have the span on the square. The bed joints of such arches must be perpendicular to the thrust along the arch, and the usual practice is that of spiral courses, because the true form is costly and difficult to execute. We will now show how the plan of a skew arch is set out in a practical form.

By reference to fig. 166, angle BAD = angle of skew. Divide AD into any convenient number of parts (equal), and through them draw parallel lines HK, etc., and draw AH perpendicular to HI, and set out ACB as the decided-upon form of soffit on the face, and then transfer such a curve to AED. produce AH to HAK, fig. 167, and from any point A draw AED perpendicular to HK, and set the distances AE, ED, etc., equal in length to the arcs AE, ED, etc. This is a somewhat tedious process, though quite simple in itself.

$$\text{arc of any circle} = \frac{\pi NR}{180} \quad (182)$$

where R = radius and N = number of degrees at centre of that arc. Such a drawing is usually done by practical engineers to a large scale. If one is handy with the slide rule, the following will be understood and will save time, viz. :—

$$\left. \begin{array}{l} A \quad 115 \quad \text{degrees at centre} \\ B \quad \text{diameter} \quad \text{length of arc} \end{array} \right\} \quad (183)$$

Then AED, No. 4, represents the development of the section on the square. Through the points of division in AD draw lines parallel to HK, then lay off from AD ordinates EC, DB, etc., and so get the curve ACB. This, then, is a vertical section on the skew; then draw similar curves at H and K, making AH = HK, etc. Then IHKL is the development of $\frac{1}{2}$ the soffit, viz. $\frac{1}{2}$ the soffit laid out flat, which is all that is required because the two halves will naturally be similar and MILN is part of the abutment.

Now having drawn a convenient number of parallel curves to ACB, they are the curves of pressure, and are the lines of the side joints, as in fig. 168.

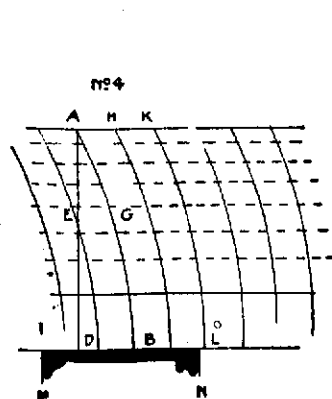


FIG. 167.

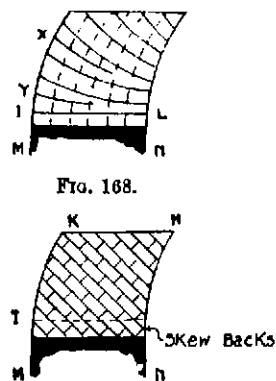


FIG. 168.

It is usual to introduce extra joints at points such as X and Y. This construction is not often used, however, for the reasons given above, its place being taken by the spiral courses shown in fig. 169. As regards deep culverts under railway banks, etc., the elliptical form adapts itself well to such a position. In this case we want to find the quantity C , explained above, and the proper value for it is

$$C = \sqrt{\frac{1 - \sin \theta}{1 + \sin \theta}} \quad (184)$$

θ being the angle of repose of the material of which the culvert is usually about 30° as before stated. If it (C) is less than the quantity stated in Formula 174, failure such as in fig. 165 may result. In such a culvert the entire ellipse may be used, but in railway tunnels only about $\frac{3}{4}$ to $\frac{1}{2}$ is used, and an inverted arch having a depression of about $\frac{1}{4}$ the span substituted. The exact form of such an ellipse would be a geostatic arch. In such culverts we make the longest radius usually equal to

$$\frac{\text{rise}^2}{\frac{1}{2} \text{ span}} \quad (185)$$

and so, by having found C , as in Formula 184, we can decide upon the minor radius, and the minimum thickness of masonry or brickwork equal to

$$\sqrt{120} \text{ radius} \quad (186)$$

same as Formula 178 for keystones. This thickness is usually retained as constant throughout. The curve in No. 4, fig. 167, is the elliptical curve to which the centering of a skew arch is set out to. In the rib shown in fig. 170 (and in any arched rib), B and B is the linear arch. Now the superincumbent load tends to depress such an arch, so that each part of the arch is not compressed in a direction parallel to a tangent to the linear arch. But it assumes the new (dotted) curve due to there being bending as well as direct compression on such an arch. When the centre of a segmental arch is not known it is easily found, as in fig. 171, the intersection arcs all having equal radii.

What is termed a stop abutment on a series of arches is an extra heavy pier at every 5th arch or so, to divide the possible area of failure into small sections if one arch fails.

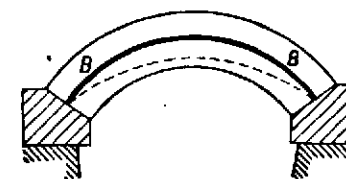


FIG. 170.

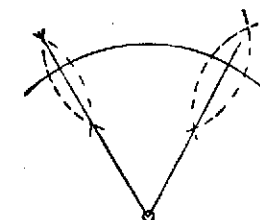


FIG. 171.

In segmental arches, if we want to find the length of the intrados, such is equal to $\frac{\pi RN}{180}$ as stated above when dealing with skew arches. Keystones in arches are not essential, though usually provided. Culverts on the skew may be built with elliptical centres and spiral courses, but for such work, which usually has no artistic point of view, it is usual to build an ordinary barrel vault and cut the end bricks on the rake, which is cheaper and quite as efficient for its purpose. We shall treat Centering in another chapter. The proper angle for skew backs of an arch is found by

$$90 - \frac{N}{2} \quad (187)$$

N being the degrees of the angle subtended at the centre of such an arch. The elliptical arch is easily set out by means of the trammel, or if such is not handy, we may construct a paper trammel as in fig. 172. First set AB = span and CD = rise, then by sliding c down the centre line and b along the springing line we trace out the curve of the ellipse. We may also draw a form of ellipse with 3 centres, 2 being of a radius equal $\frac{1}{2}$ span, and the other of $\frac{1}{4}$. Fig. 173 shows the construction. The advantage of such an arch over the true ellipse is that it only requires three shapes of voussoirs.

The following is an alternative method of setting out an ellipse (see figs. 172 A, B) :—

If a full-sized curve be required for setting out the arch, rubbing the

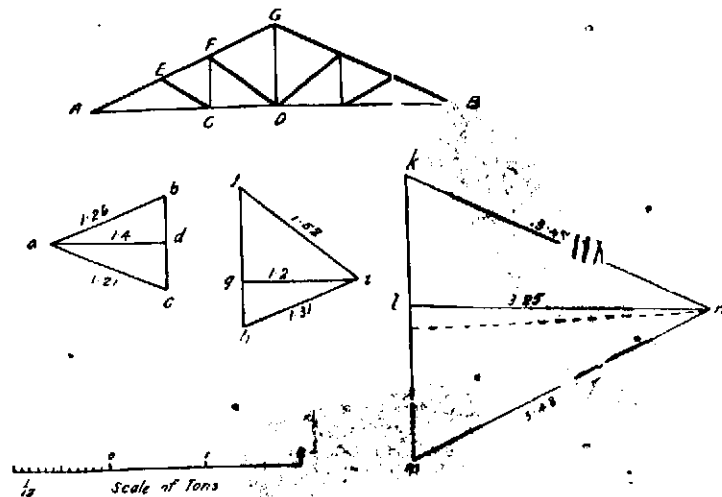
if we at the same time cut out the lower horizontal member (or boom), we can transform the structure into one of Class I., but if we were to provide the two diagonals and not remove the 3rd member, then we should bring it under Group III., when, if it was an important structure, a change of load might have disastrous effects.

Now, in all framed structures if p = number of polygons in the frame diagram, then in *Case I.* the actual number of members = $2p + 1$.

In *Case II.* this becomes $2p + 1 - d$ if d = the parts necessary to add. In *Case III.* the number is $2p + 1 - e$; and in *Case IV* the number is $2p + 1 - e + d$, e referring to redundant members.

It is also useful to remember that—

1. Lines which meet at a point in the frame diagram enclose a polygonal space in the stress diagram.



Figs. 174, 175, 176, and 177.

2. Lines in the frame diagram which enclose a space will all meet at a common point in the stress diagram.

3. Lines which represent external forces should always form a closed polygon in the stress diagram.

4. A line should represent a single force only, which means that any triangle which encloses a letter A will have its corresponding lines of force in the stress diagram meeting at a point A.

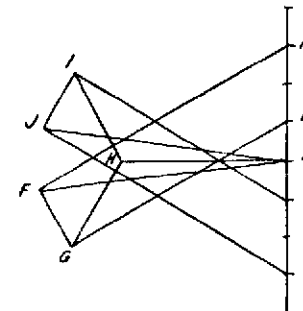
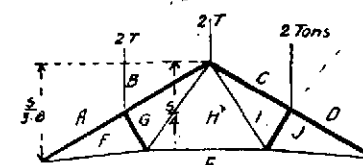
In a structure in which no roller bearings are provided the load will cause a thrust inwards of the lines of reaction on the supports, thus, especially if the ends are pinned, and the weather abnormally hot, or at least much hotter than when the bridge was erected. When rollers are provided the distortion is governed by the angle of repose. Thrusts so imposed due to load will favour the strength of the structure, but those due to temperature will have the reverse effects.

Stress diagrams for use with structures of Class II. are not of much practical value, e.g. wood roof trusses, and trusses of the queen post and trapezoidal queen post type.

In this class we supply imaginary forces, and then proceed with the diagram in the usual way, these forces being reactions to bending. In these trusses, which, as in *Case III.*, have too many parts, misfitting, even slightly, will render the structure unsafe.

We will take a simple case as an introduction, see figs. 174-176. AGB is a roof truss, as shown diagrammatically, on which we will suppose there is a dead load of 3 tons distributed uniformly. Scale this at 1 ton per inch and draw a line = 3 tons. Now, we notice that the rafter has 3 divisions. We must divide this line into twice as many divisions as the rafter contains, = 6 divisions in this case. Then in fig. 175 draw $bc = 2$ divisions. Draw ab parallel to AE , ac parallel to EC , and ad parallel to AC .

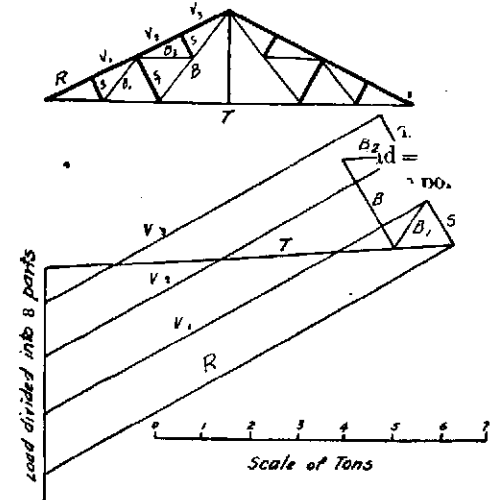
Then measure the length of these lines on the scale of tons and we have the stresses on the particular members to which the lines were drawn parallel.



Figs. 180 and 181.

necessary to be absolutely exact about such members, provided, of course, they are not too small. In figs. 178 and 179 the lettering in the stress diagram is the same as that on the roof.

These two examples are only given by way of an introduction. When applied to complex framed structures we require an effective system of



Figs. 178 and 179.

Again, make fh in fig. 176 = two divisions likewise + the distance dc , then fi parallel to FD , hi parallel to EF , and gi parallel to CD give the stresses in those parts again. Thirdly, in fig. 177 make $KM = 2$ divisions + twice fg ; from these similarly the stresses in the principals and tie rods may be found. In the same way figs. 178 and 179 show the diagram for the roof truss in question, in which case a dead load of 5 tons is assumed to be uniformly distributed. The rise of the roof truss is $\frac{1}{4}$ the span, and there is no camber in the tie rod. The diagrams shown are hardly suitable for wood roof trusses, because the joints are assumed to be pin joints, in fact they are not much used for such, as there are generally tables of scantlings for fixed spans handy, and it is not

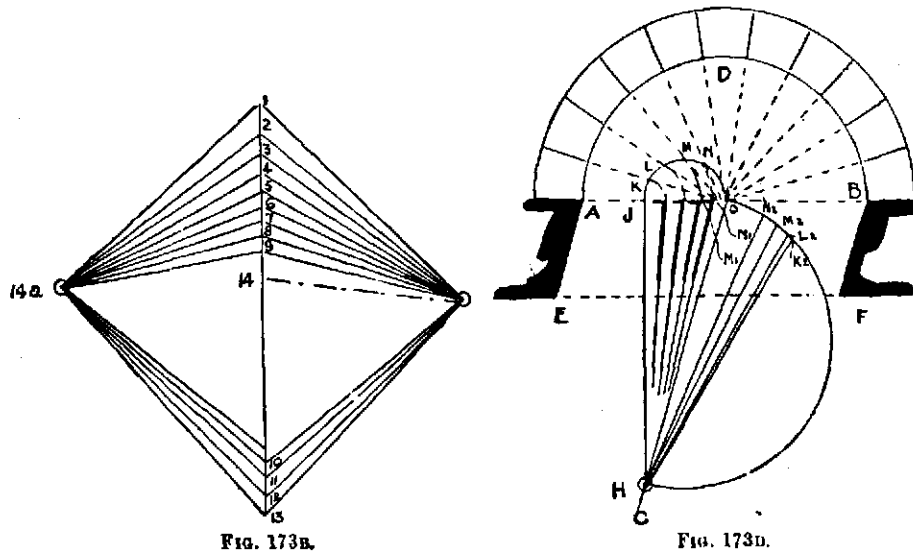
the curve of thrust. The maximum stress will occur at V, and the calculations will be

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{4550}{1.33} \pm \frac{4550 \times \frac{1}{2}}{\frac{1}{8}(1 \times 1.33^2)} = 3421 \pm 7717 = 11138 \text{ lbs.},$$

or say 5 tons per square foot compression and 4266 lbs. or 1.9 tons per square foot tension, which will be within safe limits.

Setting out Stones for Skew Arches. (Fig. 173b.)

Alternative Method.—Given the obliquity of the arch and the number of voussoirs required for its construction, the figure of the stones may be ascertained by the following graphical construction.



Ascertain the centre C of the span AB which connects with the points of division or joint lines of the arch by straight lines as shown. Again, at the point C make the angle ACG equal to the given obliquity in CG. Take any point H, and from H draw HJ perpendicular to the span line AB, and meeting it in J; on JC describe the semicircle JLNC, cutting the joint lines produced in KLM and N. Now with the distances JK, JL, etc., describe arcs intersecting JC in the points K'L'M' and N', join KII', HL', HM', and HN', then the resulting angles HK'A, HL'A, etc., will be the angles of the faces of the stones to which they refer.

In order to find the angle of the bed on HC describe the semicircle HM'C, then from C, with the distances CK, CL, etc., as radii, cut the semicircle HM'C in the points K'', L'', M'', and N'', join the points HK'', HL'', etc., then HCK'', HCL'', HCM'', and HCN'' will be the angles of the bed.

CHAPTER XII.

GRAPHIC STATICS.

THE use of graphic statics for finding the stresses in the members of engineering structures of iron, steel, and timber will be dealt with in the present chapter. In the last, there were placed before the reader certain formulæ by means of which the effects of superincumbent loads on engineering structures can be analysed. The theory of such structures is, of course, a most important part of the engineer's profession, and it is only of late years that it has received proper consideration. When the subject is studied more profoundly, very complex mathematical problems, and often long and tedious calculations, involving the use of higher mathematics, are needed. Such knowledge, although a great advantage, is not always possessed by the average engineer, because it takes years of study; moreover, it should be the aim of pupils to spend some of their valuable time in their pupilage to becoming thoroughly *practical* in their profession, in preference to being merely adept handlers of equations and formulæ. To avoid these tedious calculations, what are generally known as graphic statics, which are elaborate forms of the triangle and polygon of forces, are employed, with which, by the aid of linear diagrams drawn to certain scales, with lines parallel to the various members of the structure in question, we can read off by their length and direction the various stresses and strains in the structure. Graphic statics may be employed in reference to roof trusses, bridge trusses, lattice girders, arched ribs, domes, built-up stanchions and walls, etc. The latter will receive consideration later under the heading of Walls.

The great advantages of graphic statics are these: they are *simple* and accurate if properly drawn and understood.

The general principles of stress diagrams of framed structures may be summed up as follows.

All structures are classified under 4 groups, which may be arranged as follows:—

Group I.—Perfect structures; that is, structures in which all the joints are pin joints and when loaded no distortion would take place; and if any of the parts were made longer or shorter, no induced stresses would result.

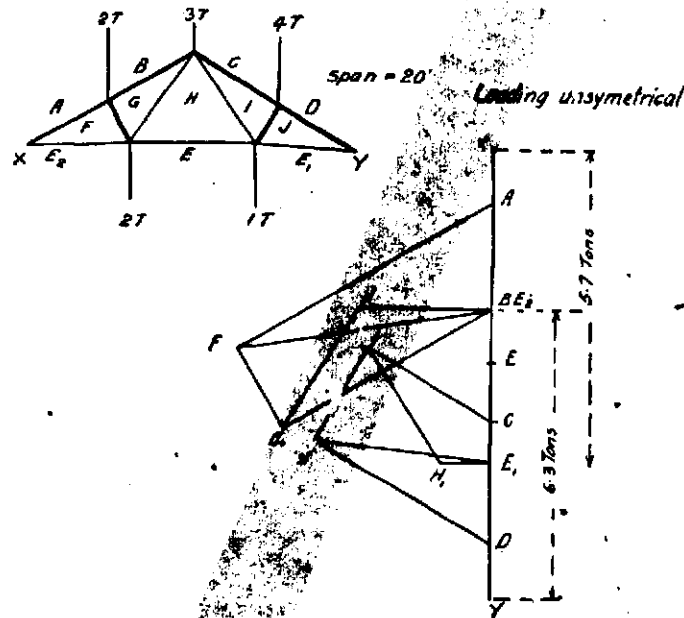
Group II.—Structures which, as before, assuming all the joints were pin joints, would only resist a change of shape under load.

Group III.—Structures which were so designed that they had too many parts.

Group IV.—Structures which have too many members in one part and not enough in another. Now, in Group II. we shall very often have a truss with rectangular bays. By the insertion of a diagonal, or even two diagonals,

lettering for all the diagram, or else we are sure to fall rapidly into confusion ; in fact, the whole success of such diagrams depends on the clearness and system of the notation. For instance, in fig. 180 the student should go all round the diagram with the letters A B C D, etc., and put E below the whole, then start in the spaces with F G H I J, and so on. Then draw the load line to any scale which may be convenient to decide upon.

Divide it into twice as many parts as the rafters have, in this case = 8 parts. We will assume the loading is symmetrical. Put the letter A at No. 2 division, B at No. 4, C at No. 6, D at No. 8, and put E at the centre. It will be seen that E is the point of convergence for a number of lines,



Figs. 182 and 183.

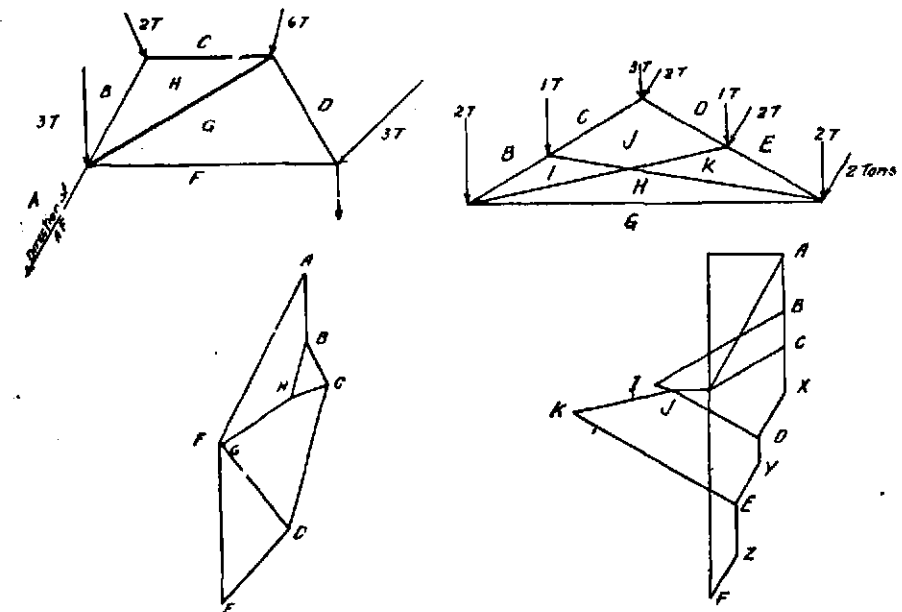
because the loading is symmetrical. Then when a line is drawn, say, from A parallel to the rafter, we call it AF, that is, in the stress diagram. AE corresponds to AF in the frame diagram, likewise BG. We would then proceed to draw EH, and from H a line parallel to GH. This will cut BG in G and so fix that point. Having G fixed, we can draw from G a line parallel to FG, cutting AF in F, and so fix the point F. In the same way we can treat all manner of stress diagrams by using a little forethought, and drawing lines which will give us fixed points from which we can draw others, and so on.

Another case now confronts us, however in figs. 182 and 183. Fig. 182 shows a frame diagram in all respects similar to the one in fig. 180. The loading here, however, is not symmetrical, there being 2, 3, and 4 tons on the rafter at the points shown, and two hanging loads of 1 ton and 2 tons, due perhaps to some light shafting or brackets being slung therefrom. Draw a load line equal to the top loads in the usual manner, that is, equal to

2 + 3 + 4 tons = 9 tons. Suppose all these loads to be applied to a beam of a span equal to the span of the roof, and at points directly, vertically under those where they occur in the roof. Now, by the principles of moments, find the reactions on each support so,

$$\frac{(2 \times 5) + (3 \times 10) + (4 \times 15) + [(2 \times 6) + (1 \times 4)]}{20} = \frac{126}{20}$$

= 6.3 tons reaction at the point Y, the last two quantities in square brackets referring to the hanging loads shown. Now our total load = 12 tons, 9 tons above and 3 tons hanging. $\therefore 12 - 6.3 = 5.7$ = reaction at the point X. Now very carefully letter the diagram as shown, putting E in the usual



FIGS. 184 and 185.

FIGS. 186 and 187.

place, and in addition the letters E_1 and E_2 . At the top of the load line put the letter X, and at the bottom put Y.

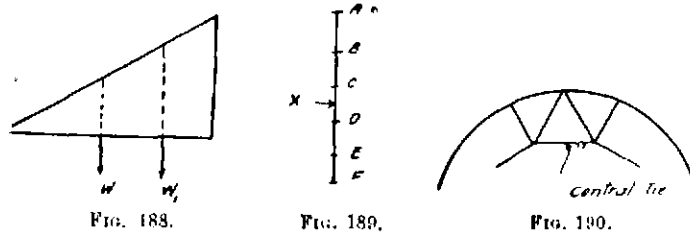
From X mark off down the load line the distance = XE_1 = reaction at X = 5.7 tons, and from Y upwards similarly the reaction at Y = 6.3 tons = YE_2 , and now, instead of a common converging point at E, we have three points E , E_1 , and E_2 , as shown, in order to get the stresses in the horizontal ties and GH and HI. It will be seen that, by comparing with the previous diagram, the extra load mostly affects the two struts and the two ties GH and HI.

It will also be noted that in most frame diagrams those members subject to compression are shown by thick lines, those in compression by thin ones. But on these diagrams it will be necessary to find out if we have pushes or pulls in the members in question. The following is the rule given by Professor Jamieson in his *Applied Mechanics*, p. 164, vol. ii.

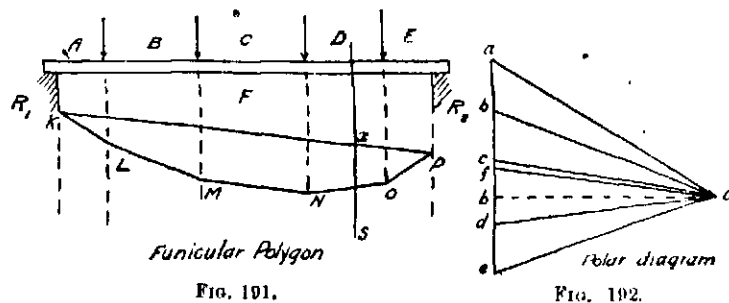
Take the letters on each side of the bar in the frame diagram in the same

order with respect to the joint on which the bar acts as we have taken the letters on each side of the external forces acting on the frame. Then along the line in the stress diagram which is named after the bar, from the first letter towards the second gives the way of the stress's action with respect to the joint under consideration. If the way is towards the joint, the stress in the bar is compression or push, and the bar is called a strut. If the way is from the joint, the stress is a tension or pull, and the bar is called a tie.

Figs. 184 and 185 show a somewhat complicated case. The loads are



not now vertical. Loads which are not vertical are usually the result of wind pressure on roofs, etc. In this case the load line is no longer vertical, but broken into units of various lengths and directions, according to the value and direction of the loads to which they are respectively parallel. ABCDE is the load line. The two lines BH and CH fix the point H, while GH and GD fix the point G similarly. We are also assuming, for demonstration, that one end of the frame rests upon rollers, like the large bridge trusses. In this case we obviously have a vertical reaction and want to find its value, but at A we have a reaction of which we know neither value nor direction. Then by



ED, having settled the point E, draw EF vertical, because we know it as a vertical reaction. Then the line FA gives both the direction and magnitude of the reaction at A.

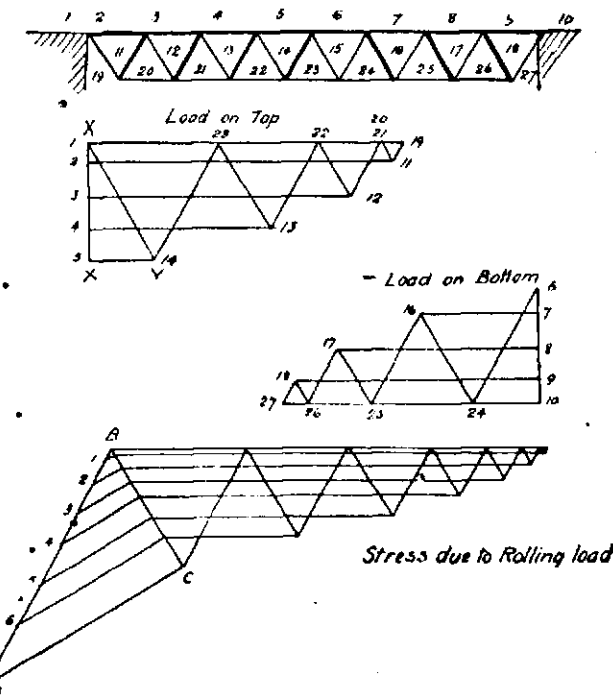
In big roofs we also very often employ a roller bearing at one end. In all cases the free rafter should be turned towards the prevailing wind (usually west in Britain), because by neglecting to do so unnecessary stresses are set up in the truss.

In figs. 186 and 187 we show the diagrams for a roof in which the wind pressure is taken into account. Such a form of roof is not suited for practical construction, but the form of the frame does not materially alter the stresses therein, as they depend merely on the load. It is assumed to be securely

anchored at both ends. We do not know the direction of the reactions, and have to find them. The reader should now be able to understand the stress diagram without further explanation.

The best way to treat hanging loads which we referred to above is to produce them vertically upwards in the same direction as that of their action, and from the points at which they act until the lines cut the rafters. Then simply treat the frame as having extra loads applied at these points. See fig. 188.

Take a roof truss having a load Sue, say, similar to fig. 189. We want to

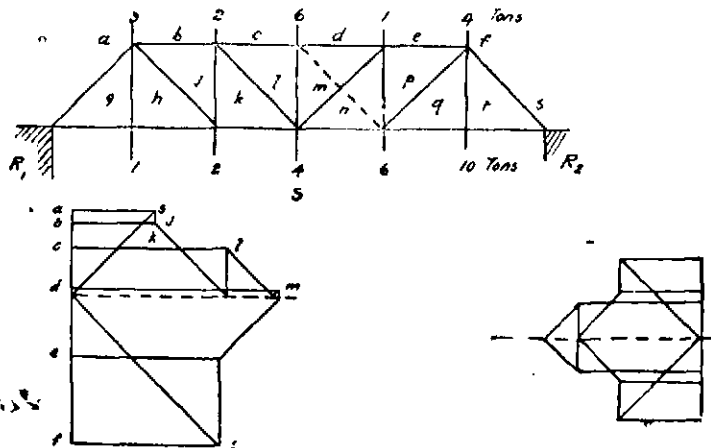


find the reaction on the supports. If the loading is symmetrical the mid-distance X between CD will give this.

Suppose, now, we have a roof truss of the bowstring type, we want to draw a stress diagram. In this case we always assume the wind pressure acting tangential to the curved surface. At the top we shall probably have a horizontal tie, as in fig. 190. It is wise to find the line which gives the stress on the rod early in the diagram. It will make the rest of the work come easier.

We will now describe what is called the funicular polygon, as applied to a loaded beam, and which, in such cases, is often called the bending-moment diagram. Take the beam loaded as in fig. 191. Let it be the same way as we have described. Draw the load line ae , and take any point O at a convenient distance from ae . This point is called a pole. Join o with

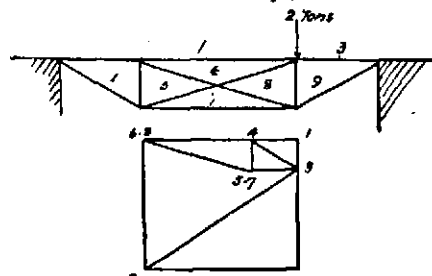
abcde. Take any point *K* directly below *R*₁, draw *KL* parallel to *ao* cutting the load line *AB* produced. In the same way draw *LM*, *MN*, etc., and complete the diagram. This diagram is then called the funicular polygon, while the diagram in fig. 192 is called the polar diagram. It has already



Figs. 197, 198, 199.

been explained that the BM at any section of a beam is the moment of the resultant of all the forces which act on one side or other of the section, then by means of the diagrams we can ascertain the bending moment at any point in a frame.

To do this we multiply the ordinate of the point by the polar distance.



Figs. 200 and 201.

In fig. 191, at the point *y*, *xy* is the ordinate and *Oh* drawn horizontal is the polar distance. ∴ the bending moment at the point $x = xy \times Oh$.

There is another way of determining the stresses in the members of a structure by what are known as Ritter's methods of sections. Its fundamental principle is that the equality of moments may be extended to any number of forces, but the sum of the moments of the forces tending to cause motion in

one direction must always be equal to the sum of the moments tending to cause motion in the opposite direction. Their treatment is almost outside the scope of this work, but interested readers would do well by referring to Professor Jamieson's *Applied Mechanics*, vol. ii. p. 623. (They are much used by designers and draughtsmen of structural steelwork.)

We cannot very well say anything more in general reference to graphic statics as applied to structures. The principle is the same in all of them; they want a little care and forethought, and, above all, proper notation. We shall now give a few more typical examples to illustrate our previous discussion

Case I. A Lattice Girder with a uniform Load on the top Boom (see figs. 193, 194, 195, 196).—On the vertical line *XX* set off to scale the units of load at each apex. As the girder has 8 divisions, these equal $\frac{1}{8}$ total load. Draw from each point so set off lines parallel to the top and bottom booms, and diagonal lines parallel to the bracing. Then the stresses can be measured as usual by the scale of tons. In fig. 196, draw *BC* equal and parallel to *XY*, and draw *BA* at any convenient angle. Make *B1* = load on 1, 1-2 = loads on 2, and so on up to point 7. Join points 7 and *C*, and through the points 3, 4, 5, 6 draw lines parallel to it to cut the line *BC* as shown. Then draw the parallel lines and start at *C*, drawing the diagonals to each of them in turn till we finish up. These lines then will give us the greatest stresses due to the maximum travelling load.

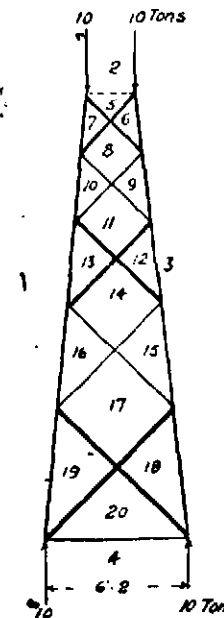


FIG. 202.

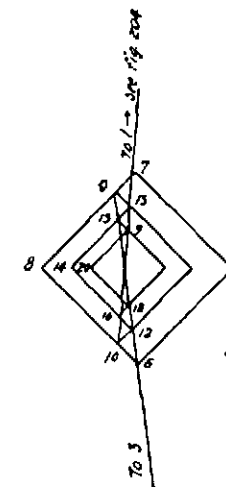


FIG. 203.

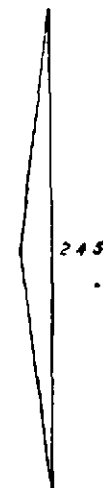


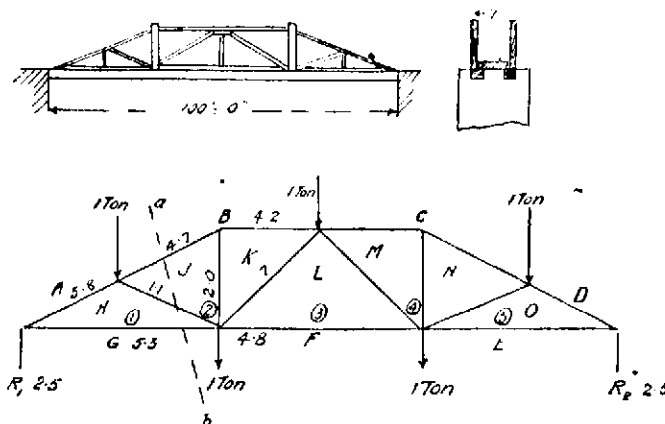
FIG. 204.

Case II. A Lattice Girder with a Load unequally distributed (see figs. 197, 198, 199).—Find the reactions on the supports in the usual manner, and lay off these loads on the vertical lines. Draw the stress lines parallel to each member as shown. Owing to the heavy loads on the right-hand side of the girder, the stress on *MN* becomes compressive. We should then, in all probability, introduce an extra member, shown by the dotted line.

Case III. A Braced Beam (as shown in figs. 200 and 201).—The diagrams are quite self-explanatory.

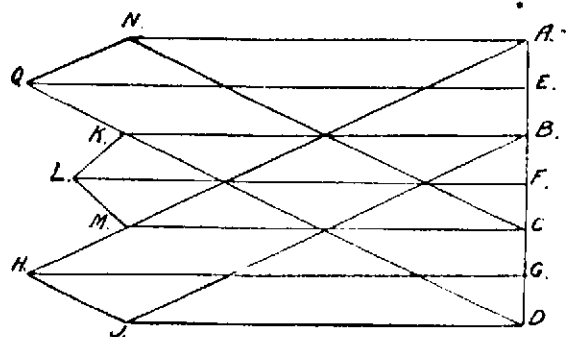
Case IV. Stress Diagram for a Lattice Stanchion.—In order to work out the stress diagram, the central load of 20 tons must be divided into two loads of 10 tons each, acting at the outer angles of the stanchion. The frame diagram will then be as shown in fig. 202, from which the stress diagram, fig. 203, may be easily worked out, while fig. 204 shows part of the stress

diagram enlarged to four times its original size. In calculating the members it should be noted that, although the member 2-5 has no stress from the reciprocal diagram, it will have to resist a BM of $\frac{WL}{4} = \frac{20 \times 2 \times 12}{4} = 120$ ton-ins. If the stanchion consists of uprights and bracing all on one



FIGS. 205 AND 206.

plane at right angles to the bracing, and if the stanchion is a 4-sided one and each side braced as shown, the corner members must be calculated as columns between the points of connection with the braces.



• FIG. 207.

As a good example of what might be an anyday occurrence in an engineer's office, we give the following worked out very fully.

Required the scantlings of a timber bridge of the form in fig. 205. The distances between girders = 7 ft., the load = 6 small waggons 6 ft. long, weighing 2 tons each, such as might be required on a large cleansing depot of a big city. Figs. 206 and 207 show the diagrams.

Timber Bridge

The type of truss shown is more suitable for somewhat smaller spans than 100 ft., but the stresses and scantlings are worked out on the assumption that the type must not be altered. The main objection is that the top boom, where it slopes to the abutment at either end, is 37 ft. 6 in. long, and has to resist in compression the maximum boom load owing to the shallow depth of the truss at the ends. To avoid using such a heavy stick of timber as this condition would require it will be necessary to use portal bracing to support it at mid length; this, again, will only leave a headroom on to the bridge of half the depth of the truss, say 8 ft. If the waggon will clear this, it will be all right; but if not, either the heavy section of timber must be used or the type of truss altered. It will be necessary to use the same scantling of timber right through the top boom, so that this strut practically fixes the section of the boom. The wide bays also cause a large bending moment on the bottom boom, while shorter bays would reduce it.

The quickest way to arrive at the dead load stresses will be by a stress diagram, while to obtain the live-load stress it will be necessary to draw a separate diagram for each position of the live load. As, however, we can tell by inspection the position to place the live load in to obtain the maximum stresses, it will be quicker to place the load in the required position and obtain the stresses by calculation. Draw first a frame diagram, assuming a central depth of 16 ft., and making all bays equal and 17 ft. long; this gives a total length of $17 \times 6 = 102$ ft. as the length of truss, centre to centre of supports, which allows 2 ft. landing at each end of truss. It is only possible to assume a dead load, and then, when all the sizes have been got out, a correct estimate can be made of the weight of the whole bridge, not forgetting the deck, rail bolts, connecting plates, etc. If this actual weight be much greater than that assumed, a fresh design must be got out, assuming a new dead load greater than the calculated weight, as the increased size of the numbers under the new conditions will still further add to the weight. We will assume the dead load at 1 ton per bay. The vertical members omitted from the frame diagram simply act as suspenders for the load on the bottom chord, and transfer the load direct to the top boom.

After lettering the diagram as shown—i.e. between each external force in each space—the stress diagram may be drawn. Draw the vertical line AD, and beginning at the top, set off the loads in the direction in which they act. Thus AB, the first load, is set off downwards, also BC and CD, while DE is set off upwards, as it represents the reaction, then EF and FG downwards, and finally GA upwards. Lines are now drawn parallel to the respective lines in the frame. The lines in the force diagram measured by the force scale, to which the loads were set off on AD, give the stresses in the corresponding lines in the frame diagram. Thus, AH in force diagram scales 5.87 tons, and equals the stress in AH on the frame diagram. The omitted suspenders are each subject to a tension of 1 ton—rather less, really, as about $\frac{1}{2}$ of the dead load is really due to the top half of the girder; but it is not necessary to allow for this in a small girder. The live load of 6 ft. by 6 = 36 ft. long = $6 \times 2 = 12$ tons = 6 tons full girder. It so nearly covers two bays that we may take the panel load as 3 tons. The maximum stress in HJ will occur when the head of the train has advanced halfway along the second panel: the panel point at G has then a maximum load of 3 tons. The reaction at $R_2 = \frac{3}{2} = 1.5$ ton, and this is the shear on HJ. Then $0.5 \times \text{cosec}$

ϕ = stress, or $0.5 \times 2.29 = 4.58$ tons, where ϕ = angle HJ makes with horizontal = $25^\circ 48'$. By loading panel points 2 and 3 the maximum shear is obtained on KI, and also with the load coming on the reverse way on LM. The reaction R_2 here = $\frac{2}{8} + \frac{3}{8} \times \frac{2}{4} = 2.5$ tons. The stress KL and LM = $2.5 \times \text{cosec } 43^\circ 16' = 2.5 \times 1.46 = 3.65$ tons. The maximum stress in JK occurs when panels 2 + 3 are loaded; it equals the vertical component of the stress in BJ, so that we must know this before we can calculate JK, as it is impossible to take a section anywhere through the truss, including JK, where there are not four unknown forces. Pass a section through the line $a \dots b$, cutting three members.

By taking moments about the point of intersection of two of the members we find the stress in the third. Thus $R_1 = \frac{2}{8} + \frac{4}{8} \times \frac{2}{4} = 3.5$ tons. Then stress on BJ = $S_{BJ} \times 14.4 = (3.5 \times 34)$, $S_{BJ} = \frac{3.5 \times 34}{14.4} = 8.2$ tons.

The stress on JK is therefore $8.2 \times \sin 25^\circ 48' = 8.2 \times 0.435 = 3.5$ tons. The maximum stress in the top boom occurs in the member AH, and is a maximum when panel points 1 + 2 are loaded. The reaction $R_1 = \frac{2}{8} + \frac{4}{8} \times \frac{2}{4} = 4\frac{1}{2}$ tons. $\therefore S_{AH} = 4.5 \times \text{cosec } 25^\circ 48' = 4.5 \times 2.29 = 10.3$ tons. The maximum live-load stress in the bottom boom occurs in the part GH and is a maximum when apices 1 and 2 are loaded. It equals the reaction $R_1 \times \cotan \phi = 4.5 \times 2.06 = 9.27$ tons. The live-load diagram is drawn the same way as the dead. That illustrated shows the live-load stresses when apices 2 and 3 are loaded. The other diagrams may be constructed in the same manner. We now have

$$\begin{array}{r} S_{AH} = 5.8 \text{ tons dead} \\ 10.3 \text{ ,, live} \\ \hline 16.1 \text{ ,, total} \end{array}$$

Allowing for the ultimate strength of the timber in tension 8000 lbs. and in compression 5000 lbs., the safe loads with a factor of safety of 8 are 1000 tension and 750 compression. Let l = length of member in inches; r = radius of gyration = minimum width $\times 0.289$; c = constant = 3000; S = safe load on member in pounds per sq. in. Assume 12 in. by 12 in. timber, $r = 0.289 \times 12 = 3.4$. Then

$$\begin{aligned} S &= \frac{750}{1 + \frac{l^2}{cr^2}} = \frac{750}{1 + \frac{222^2}{3000 \times 3.4^2}} \\ &= \frac{750}{2.36} = 318 \text{ lbs.} \quad \frac{318 \times 144}{2240} = 20.4 \text{ tons.} \end{aligned}$$

$$\begin{array}{r} \text{The stress on HG} = 5.3 \text{ tons dead} \\ 9.2 \text{ ,, live} \\ \hline 14.5 \text{ ,, total.} \end{array}$$

Assume as size 12 in. by 10 in. $\frac{W}{A} = \frac{14.5}{120} = 0.12$ ton. The bending moment from live load = $\frac{WL}{10} = \frac{3 \times 17 \times 12}{10} = 61$ in.-tons. Section modulus = $\frac{BD^2}{6} = \frac{12 \times 100}{6} = 200$. $\frac{BM}{\text{mod.}} = \frac{61}{200} = 0.3$. $\therefore$ total stress = $0.3 + 0.12 =$

12 ton = 940 lbs. per sq. in. The size assumed is sufficient to meet the stresses, but 12 in. by 12 in. is better, as an allowance must be made for mortise holes and a joint.

$$\begin{array}{r} \text{The stress on AH} = 4.58 \text{ tons live} \\ 1.10 \text{ ,, dead} \\ \hline 5.68 \text{ ,, total.} \end{array}$$

The length = 19 ft. For the sake of the joints the width should be the same as the top and bottom members.

Try a section 12 in. $\times$ 8 in., $T = 0.289 \times 8$. Then

$$S = \frac{750}{1 + \frac{(19 \times 12)^2}{3000 \times 2.3^2}} = \frac{750}{1 + 3.0} = 180 \text{ lbs.}$$

Area 12 in. by 8 in. = $\frac{96 \times 180}{2240} = 7.7$ tons. This is slightly stronger than required.

$$\begin{array}{r} \text{The stress on KL} = 3.6 \text{ tons live load} \\ 0.7 \text{ ,, dead load} \\ \hline 4.3 \text{ ,, total.} \end{array}$$

The same size timber may be used as for HJ, or it may be slightly reduced desired.

$$\begin{array}{r} \text{The stress on JK} = 2.0 \text{ tons dead} \\ 3.5 \text{ ,, live} \\ \hline 5.5 \text{ ,, total.} \end{array}$$

$$\frac{5.5 \text{ tons} \times 2240 \text{ lbs.}}{1000 \text{ lbs. per sq. in.} \times 12 \text{ in.}} = 1 \text{ in. wide.}$$

2 in. by 6 in. would be a suitable size, and the same for the other verticals. diagonal braces should be used in the top bracing and also in the deck to resist the wind stresses. The wind will not affect the boom section very much, as the maximum wind stress is in the middle of the bridge, and the boom stress is less there than at the ends. The vertical posts are subject to bending from the wind, but they are ample in size. The end member AH will be most affected; but to enter fully into the wind stresses would take up too much space.

A plate containing several examples of stress diagrams appears opposite, which should be carefully studied and worked out by the reader.

The chief place of manufacture is the valley of the Thames and Medway, where the alluvial deposit seems to have special qualities. The brands are legion, and it would be impossible for obvious reasons to describe any one manufacturer's brand as being in any way superior to another, but the author has for many years used Johnson's cement, and always found it reliable.

The average proportions of cement may be taken: clay 3 volumes to 1 volume of chalk; sometimes, however, the chalk may contain clay, and the mixture has to be modified accordingly. For testing quantities of cement on large contracts, Fajja's machine is almost universally used.

Ordinary cement of the best quality is usually heavy; and a heavy cement is a slow-setting cement, which for heavy work is very desirable. It should be of a blue-grey colour, and show a compact dark fracture. The usual weight is 110 lbs. to the bushel, and the amount of water required in mixing about 3 gallons per bushel. When brown, it shows that the proportions in manufacture were not properly gauged, and that an excess of clay was present; whereas if it is very blue, we may infer that it has been over-limed, and that it is liable to blow. A cement which will blow is not properly cool, viz. it is a "hot" cement. By plunging the arm full length into a bag of cement an experienced engineer or clerk of works can tell by the feel if the cement is "hot." The temperature should appear as near as possible the same as the arm if the quality is good.

Too much lime in a cement unfits it for sewer work, owing to the chemical action of the sewage destroying the carbonate of lime. The liability to blow may be tested for by seeing if the cement cracks round the edges after immersion in water for a day or so. Also, if a bottle is filled with neat cement and the bottle cracks, that cement will be too hot; if, however, a shrinkage takes place, it is under-burnt. Some engineers use a hot cement in under-pinning work, because when the concrete is rammed well in the expansion due to the hot cement causes the whole to consolidate very considerably.

Cement must always when stored be emptied out of the sacks and put 2' 6" deep on to a floor of wood; in a dry weatherproof building this will cause it to increase in bulk. A cask of cement contains $3\frac{1}{2}$ bushels, while a sack contains 2 bushels and weighs 2 cwt.; and for mixing 6-1 concrete we use a box 5' x 3' x 1' to one bag of cement.

Roman Cement, also called *Melina Cement*, is a natural cement made from the septaria nodules found in London clay and elsewhere. These are burnt, and by such they lose $\frac{1}{3}$ of their weight; on grinding, however, they increase again by $\frac{1}{4}$. It is usually used neat, principally for sewer work and for work in which very quick setting is required. For joining sewer pipes it is unequalled.* It loses strength if mixed with sand, and also has the disadvantage of disintegrating with age. The tensile strength at the end of seven days is about 90 lbs. per sq. in. It should not weigh more than 75 lbs. per bushel. If it does, it may be concluded that it is either stale or over-burnt, two undesirable qualities. The colour is a rich brown, similar in many respects to cocoa, but not quite so dark.

Concrete (see also Chapter on Concrete Steel).—This is the term applied to all mixtures of a semi-liquid state used in building construction which have the property of hardening after a period which varies with the material. There are two essential parts, the *aggregate* and the *matrix* or binding

* This statement is open to criticism, as many leading engineers now use Portland, especially with Stanford joints.

material. The aggregate is best made from porous material, broken up into small angular fragments. Slag from iron furnaces, if not too vitreous, is very suitable. When large stones are inserted during the laying of the concrete they are termed *displacers*. They must always be wetted before setting, and never touch one another, but be completely surrounded with the concrete. For concrete flags, shingle should be used as aggregate and very little sand added. It must be well washed, like sand, especially if from a sea-beach, to rid it of any salt, which, being hygroscopic (that is, possessing an affinity for moisture), will cause dampness in the structure, a disastrous result in house walls.

The proportions of concrete vary. The usual is 6-1 sand or gravel and best Portland cement. The sand should, if possible, be clean, sharp, well-washed river sand, free from loam or earthy matter. Pit sand is not so good, although extensively used, while sea sand is best avoided altogether, for the reason stated above. When the concrete is liable to exposure to heat a small proportion of gypsum may with advantage be added. Concrete made with hydraulic lime is termed *Béton*.

When specifying concrete the following points should be borne in mind:—

"All concrete work to be in proportion of — to 1 best Portland cement of approved quality, and clean, sharp sand or gravel. The materials to be thoroughly mixed on a platform and watered through a rose, and turned twice over before use. Separate boxes to be provided for measuring the exact proportions, 1' 8" x 1' 0" x 1' 0" and 3' 0" x 2' 4" x 1' 0". All concrete must be well rammed between casings, laid continuously to give thorough bonds and wetted between lifts."

Breeze concrete is sometimes used, in proportions of about 6-1; it is only fit, however, for internal walls. It has the convenient property, however, of holding nails with great tenacity.

It is an advantage, when specifying, to say, instead of a 5-1 mixture (or any other proportion), "so many bushels of cement per cubic yard of mixed concrete," and so ensure more exact proportions. These proportions must be nearly double if hydraulic lime is used. What is termed *Thames ballast* is two parts of gravel to one of sand. Always specify concrete work to be finished "hard," and if required to a "hard, smooth surface."

Stones.—All building stones must withstand atmospheric influences. Stones of which carbonate of lime and carbonate of magnesia form a large part easily decay when subjected to sulphurous gases, carbon dioxide, and hydrochloric acid, while oxygen will destroy stone which contains much iron. The rainfall also influences the decay, and likewise high winds, which have a very grinding effect.

Working downwards in a Portland stone quarry we find—(i.) Roach, (ii.) White bed, (iii.) Curf, and (iv.) Base bed. The two latter, which do not weather well, are not used for outside work. The quarry sap should always be dried out by seasoning. Engineers abroad, in hot climates, however, do not approve of this procedure. All stones must be laid on the natural bed, but in arches the stone is laid at right angles to the thrust along that stone. Lichens have a destructive effect on stones, and many marine organisms attack stone. Granite, however, will withstand the effect of the latter. On testing, stones should absorb very little water, as a stone which is able to soak up much water will be sure to fail during a heavy frost. York paving slabs are very prone to this defect. When soaked in dilute hydrochloric or sulphuric acids, carbonate of lime or magnesium will, if present, cause effervescence.

The three chief varieties of stone are siliceous, such as flint; argillaceous,

CHAPTER XIII.

MATERIALS OF CONSTRUCTION.

THE following account of the structure, properties, and component parts of various materials used by builders, civil engineers, and others engaged upon works of construction is intended to afford to the users of such materials, and to the student of architecture and civil engineering, a clear and comprehensive insight into the natural composition and structure of those materials, their essential properties, and their manufacture (when not found in nature), as well as to explain the various defects to be guarded against, and the methods of testing and selecting specimens in everyday use.

Such materials naturally differ in composition in various parts of the country, and the actual figures given are in no case to be taken as binding,

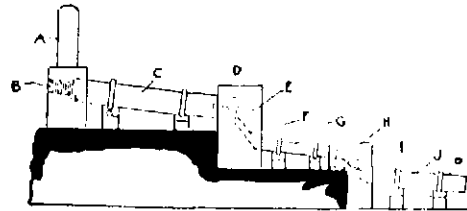


Fig. 208.

but as averages only. Engineers vary considerably in their methods of testing, and much diversity of opinion exists as to the suitability of certain materials to certain purposes.

Cement.—Of late years what is known as *Portland cement* has largely taken the place of so-called hydraulic limes, which were principally made from blue lias limestone, and will be described later. Portland cement is made from chalk and clay principally. These two are first reduced to "slip," that is, the chalk and clay or river mud of a suitable variety are mixed with water in large shallow tanks or lagoons and allowed to dry. When all the superfluous water has run off we have what is termed "slurry," of a consistency of clay, which being dried, usually by artificial means, is reduced to powder. This is then fed into a rotary kiln (figs. 208 and 209) at the chimney end, and being revolved on this it travels down to the lower end, where it meets a flame produced by coal dust in a jet of compressed air, and is by this means reduced to clinker. It is this process which requires most careful carrying out, as over-burnt or under-burnt cement has defective qualities, which will be pointed out later. Hence a happy medium is

irable. The clinker is then ground to the desired fineness, the degree of fineness being always specified in the case of *Portland cement*. It may here be stated that the term *Portland* is not, as some suppose, due to cement being originally made in Portland, but to its resemblance, when set and of a good quality, to *Portland stone*.

The ways of specifying cement are various, each engineer having his own favorite mode of testing and specifying. A good general specification, which usually ensure a good, even quality of cement being delivered by the



Fig. 209.

- | | |
|-------------------------|-----------------------------|
| A, Chimney. | F, Shoot. |
| B, Feed conveyor. | G, First cooling cylinder. |
| C, Burning cylinder. | H, Water spray. |
| D, Plumet of coal dust. | I, Crushing rolls. |
| E, Burner. | J, Second cooling cylinder. |

manufacturer, is as follows, and has been in constant use in the author's office many years:—

The cement shall be of the very best quality, and subject in every respect to the approval of the surveyor.

It shall be packed in strong, perfectly sound sacks. Any torn or hard sacks will not be accepted.

The cement shall be subject to the following tests:—

It shall be very finely ground, so that not more than 5 per cent. by weight shall be stopped by a sieve with 76 by 76 meshes per square inch; it shall have a specific gravity of not less than 3.1. Samples made into briquettes and immersed in water

hours after being made, and remaining immersed until tested, shall have an average tensile strain, 7 days after being made, of not less than 350 lbs. to the square inch; and 28 days after being made, the average tensile strain of the briquettes then tested shall be at least 20 per cent. more than the average of the 7 days' test. The cement shall not set in less than one hour, and shall be considered set as soon as no pressure can be made on it with the thumb nail. Other tests may be applied at the surveyor's discretion.

Pats of cement made with water, and allowed to set for five hours in a damp atmosphere, at a temperature of 100° F., and afterwards immersed for 19 hours in water at the same temperature, shall show no deterioration.

If the cement does not come up to these tests, it shall be removed from the store by the contractor, and at his own expense, within 14 days of written notice to that effect being given to him by the surveyor.

Samples for testing to be taken from the consignments, at will, by the surveyor or representative, who shall have the tests made, the cost for doing which shall be paid by the contractor.

Naturally, when the contract is a large one, the specification is longer and more exacting, but for general purposes in this country the above is all that can be desired. In manufacture, the proportions of clay and chalk or lime are very variable, and no standard proportions can be quoted.

See also the *British Standard Specification*, Messrs. Crosby, Lockwood & Co., price 2s. 6d.

such as slates; and calcareous (limestones). In practice, the varieties which chiefly occur are granites, slates, sandstones, and limestones.

Slates.—Slate is what is termed an argillaceous rock. Good slates should be hard and tough, give a sharp metallic ring when struck with the knuckles, have a fine grain, be non-absorbent, of uniform colour, and free from patches, veins, iron, or cross grain, and have smooth, even surfaces. The best slates come from Wales (Penrhyn), Westmoreland, Cornwall, and, in Ireland, from Killaloe and Valentia. Slates which have a greasy touch should be rejected. They should in no case absorb more water than $\frac{1}{20}$ of their dry weight. Composition nails are the best to use for securing slates; the actual proportions of metal vary, but they are usually some alloy of zinc, lead, copper, or tin. The composition should in all cases be guaranteed.

Granite consists of quartz, felspar, and mica. Much quartz makes granite hard to work, while an excess of mica is a source of weakness. What is known as *syenite* (found near Land's End) is a species of granite containing a large proportion of hornblende. Hornblende is a dark green, hard, lustrous mineral. Granite is an igneous rock, that is, one which has been affected by continued heat and pressure. Aberdeen granite is perhaps the best known variety and is of a grey colour, while those stones from Peterhead and Shapfell (also well known) are red in colour, this being due to the felspar being stained by the iron oxides. Other varieties are Mount Sorrel, and in Ireland, Newry, Killiney Hill, and Donegal. Kingstown harbour, near Dublin, is made entirely of huge masses of granite from Killiney Hill, about 3 miles distant. In practice, good granite will stand 15 to 25 tons pressure on the sq. ft.

Serpentine is a variety of granite used for mantelpieces and other ornamental work, as it is capable of taking a fine polish. Connemara marble, so much seen in Ireland, is a variety of serpentine. Serpentine is what is known as a trap rock.

Basalt, which is a very hard stone, and has the appearance of whinstone, is used for setts, and syenite is also used for the same purpose; it is even harder than granite.

It may be generally stated that stones fail owing to the effect of various acids on the carbonate of lime which most stones contain.

The "weathering of a stone" may have three distinct meanings, namely—

1. Working a window-sill to a sloping surface (verb).
2. That surface so made (noun).
3. The good property of all stones (noun).

Limestones are either pure carbonate of lime, or carbonates with a mixture of a certain proportion of sand and clay. They make very good paving setts and absorb very little water. When much clay is present carbonates become *marl*, and much magnesia present produces *dolomite*. All limestones dissolve readily in dilute acids, giving off carbon dioxide. *Bath* and *Portland stone* are the principal varieties used for building.

The average weight of Portland stone is about 135 lbs. per cubic foot, or $14\frac{1}{2}$ cubic feet to the ton. It is true Roach, which has what is termed the "screw" fossil, and the stone which has this is much prized for all engineering work. *Kentish rag** is a limestone variety having a blue-grey colour. Bath stone is white to yellow in colour, and is soft on quarrying, and liable to much quarry sap. It will, however, harden on exposure to the atmosphere. *Red limestone* comes principally from Mansfield in Northamptonshire and Caen in France.

* Many are of the opinion that this stone is a calcareous sandstone, and not a limestone.

Sandstones consist chiefly of quartz, silica, carbonate of lime and magnesia, alumina and iron oxide. The grey tint in some stones is due to the presence of carbonate of iron, and the red colour to ferric oxide (FeO_2). Green and brown tints are due to the presence of ferric hydrates. The best sandstone variety is undoubtedly from Craigleith, near Edinburgh; then comes *York stone* and *Bilston*. Kentish rag is what is termed a calcareous sandstone, although some are of the opinion that it is a siliceous limestone. Sandstone, if of a sufficiently hard variety, will make good paving stones, notably the York stone for flags. *Whinstone* is a hard, tough, unstratified variety. Another sandstone variety is the Bramley Fell stone, and the Forest of Dean in Gloucestershire produces a greyish blue variety which is much used in engineering works. In all cases it may be said that the fine-grained sandstones are used for building, and the *grit stones* for engineering purposes. A simple test for quality of sandstone is to put some pieces in a glass of water and leave them to stand for about half an hour; subsequently agitate the whole. The less resulting cloudiness, the better the stone.

Sandstone, to be of a good quality, should not weigh less than 130 lbs. per cubic foot.

When preservatives of stone are used, it is necessary to thoroughly clean the stones before applying them. The usual materials for the purpose are tar oil, linseed oil, and potash silicate (soluble glass), and the whole object is to fill up the pores on the surface and to prevent admission of the air to the interior of the stone, which would cause disintegration. The use of stone preservatives is not, however, very general.

Bricks. All clay to be used in the manufacture of bricks must be free from iron pyrites, as these will crystallise and split the brick to pieces while burning in the kiln. A small percentage of lime is an advantage, because it will tend to bind the brick together and cause it to retain a true shape, but too great an excess will have just the opposite effect, and cause it to melt and lose its shape. It is often necessary when selecting clay for brickmaking to ascertain whether it contains much lime. If we heat it with sulphuric acid, the lime will reveal itself by causing effervescence.

The composition of bricks varies considerably according to the quality of materials used, mode of burning, moulding, colour, and the purpose for which the brick is to be put. Pure clay which is free from iron will burn white, but chalk is usually added to produce white bricks. Blue bricks, known as Staffordshire bricks, are made by intense heat in the burning, which converts the red oxide of the iron into black oxide, but manganese will also cause this, and that is the secret of many imitation Staffordshire bricks now in the market, and lacking the requisite hardness of such when properly manufactured. Lime and iron together will yield a cream colour.

When bricks are dried out of doors the process is called "hacking"; when they are partly dry they are placed diagonally in rows. This latter process is termed "scintling," while the hollow recess found in most bricks is termed the "frog." It is laid uppermost by the bricklayer. Good bricks should be free from cracks, flaws, lumps, stone, and particularly from naked lime. They should be hard, well burnt, and ring well when struck against one another. Good brick clay will contain 20% of alumina, 60% of silica, and 20% of iron and other impurities. The best white bricks come from Suffolk. "Compass" bricks are those which taper in one direction, sometimes two; while "Bull-heads" have a constant thickness and varying breadth.

Firebricks are made of clay which is nearly pure hydrated silicate of

alumina. The presence of iron will spoil it, but a small quantity of lime potash and soda is not disadvantageous. When used for boiler setting, all firebricks should be true and square, not chipped at the edges, and all joints should be as thin as possible; the finished work, above all things, must have a smooth surface. The joints in such work are made by dipping the bricks into a "slurry," formed by soaking fireclay in water, a process which must continue for at least 24 hours. The clay is first soaked through a $\frac{1}{4}$ -in. sieve. All the brickwork must be thoroughly dry before firing. Any repairs to boiler setting should be made good as much as possible by actual brickwork. What cannot be so done must be very carefully hand puddled with fireclay or ganister, and a special fire lighted to set it hard. Bricks, according to the district they come from, are *loam*, *clay* or *marl*, shale or other suitable material.

Loam is clay and sand in a mechanical mixture, while marl is clay and carbonate of lime in a similar composition. Its use is restricted by the previous remarks in connection with lime in bricks. Clay will absorb water, and is swelled in so doing, and this water is only driven off at high temperatures. The presence of alkalis or acids tends to cause the bricks to soften.

It is only the refractory (infusible) clays which are used in brickmaking. The first process is tempering, whereby the clay is mixed with about half its volume of water. Good bricks when broken should show a more or less glassy fracture, free from holes. They should not crush below 1100 lbs. per square inch, and good brickwork in cement should stand 800-1000 lbs. on the square inch. Pressed bricks are made by pressing dry powdered clay with a pressure of about 5 tons per square inch and baking; bricks for use in foundations should be carefully burnt to the stage of incipient vitrification, and if for paving purposes, made of a very plastic clay. Brick earth is usually a sandy or loamy clay; pure clay, called "foul" clay, is too "strong" for brick-making, and bricks made of it will be sure to warp and lose shape. The best average proportion is $\frac{2}{3}$ of silica, $\frac{1}{4}$ of alumina, 1% of iron oxide, and $\frac{1}{2}$ % of carbonate of lime, the remainder being principally organic matter. Clay itself is hydrated silicate of alumina, and the hardening in burning is due to the water being driven off, the clay itself being partially vitrified, the presence of the lime assisting the process. Salt, for the reasons before stated in reference to sea sand in concrete, causes bricks to be damp when built into walls. In the same way, mortar made with sea sand has the same effect.

Sand-moulded bricks usually keep a better shape than the "stop" bricks. What is termed "close clamping" is the process of mixing bricks into bricks, so as they contain the requisite material in themselves for vitrification. "Open clamping" consists of laying the bricks with a layer of coke breeze and small coal between the courses. The whole is built up into clamps, surrounded with bricks, and set fire to. "Brindling" is the marking seen on bricks, viz. alternate stripes of light and dark colour, due to the bricks "crossing" in the kiln, the full play of the heat causing the extra light colour.

The clay from which bricks are made is in all cases exposed to a winter's frost before being worked. Burrs and clinkers (viz. over-burnt bricks) are found near the "live holes" of the kiln. "Dust bricks" are a very hard paving brick in which coal dust has been worked into the clay instead of sand. Glamorganshire Dinas bricks are the best variety of firebricks; 1% of lime is usually added to the clay from which they are made. *Gault* is a blue clay which makes hard white bricks. When sand is added to the gault they

become *Suffolk whites*, a soft brick very suitable for rubbers. *Glazed bricks* are usually made from fireclay. A thin coating of china clay is put on the surface, which in burning becomes vitrified. They are always set in Portland cement, and afterwards pointed in any of the usual ways with Keen's cement.

Kiln-burnt bricks invariably have a better shape than those from clamps. "Malms" are the best clamp bricks. They are made from an artificial marl which is composed of about 75% of clay, 15% of cinders, and 6% of chalk. This mixture is reduced to a slurry, which is afterward dried in the air and hand moulded. "Best" Malms are used for gauged work, "Seconds" for facing bricks, and "Pickings" for inferior work. The "best" bricks are of a light buff colour, which gradually darkens, according to quality, down to brown in the inferior "pickings." "Shuffs" are bricks full of small cracks or "shakes." Wire-cut bricks have no "frogs." They have, however, the good quality of very sharp arrises. A "rod" of brickwork = 4352 bricks when set in mortar; if dry, 5370 will be required. It is usual to supply 4500 bricks per rod, to allow for wastage and cutting.

Sand-Lime Bricks.—Clay bricks have long been used for structural purposes; and while they have given splendid service, they lack some of the important advantages possessed by some other kinds of bricks.

Among these the sand-lime product has claim to recognition. Sand-lime bricks have long been used in some countries, particularly Germany, and it appears that when the materials are first-class and the manufacture of the brick is understood, the result is generally satisfactory.

These bricks work well in structures, have a considerable carrying capacity, and will withstand a great heat. They are also waterproof.

For residence work they seem to have no fault except that they are more difficult to lay than clay bricks, because they do not have the same suction. In factory buildings they have been a failure, because the vibration from the machinery and the slamming of heavy doors causes the mortar to loosen. Even a rich cement mortar does not hold them as well as it does other bricks. Sand-lime brick is composed of good lime and clean sand, or some other aggregate, such as ground marble, crushed granite, and kindred materials. These are carefully mixed and pressed. Perhaps the greatest difficulty in the manufacture of the brick is in the proper selection of the lime. The best lime for the purpose is rich in magnesia, and it must be fresh. It is crushed, the sand is added, and the mixture is run into the press in uniform amounts. Special machinery is employed. The bricks pass from the powerful press into a form of kiln termed a "hardening cylinder," where they remain from eight to twelve hours, and in which the materials are fused. The bricks can be used as soon as they are cool. The above is a mere outline of the process, there being a number of technical details demanding great care.

Lime Sand Cements.—For producing limes we have two forms of kilns.

1. *The Tunnel Kiln*, fig. 210, or continuous-running kiln, is built of masonry and lined with firebricks. The limestone, which should be damp, is put in at the top with alternate layers of coal and brushwood. The usual size is 16 ft. deep $\times$ 4 ft. square at the bottom and 9 ft. square at the top. It is economical of fuel, but does not produce an even equality of lime like the—

2. *Flare Lime Kiln*, fig. 211, which, however, is not a continuous kiln, and uses more fuel; but it has the advantage of being easily looked after and produces a very even quality of material.

Lime varieties are—

1. *Quicklime* for mortar.
2. *Fat lime* for plastering.
3. *Hydraulic lime* for work on damp sites, for which latter purpose the blue lias lime is the principal variety.
4. *Stoke lime*, used for building walls.

The composition of ordinary mortar is 2-1 clean sharp sand and best grey limestone from Mertsam, Halling, or Dorking, or any other approved brand. For brick on edge coping, and in fact any other work in which the architect considers it advisable, 2-1 Portland cement and clean sharp sand should be used. For tall chimney shafts, 1 of blue lias lime to 3 of sand is the correct proportion, and for flat pointing 1 of lime to 2 of sand is used. A load of mortar = 1 cubic yard, and requires 9 bushels of grey lime, 1 cubic yard of sand, and enough water to make a stiff paste. It must on no account be used fresh, but subject to the usual process of tempering, that is, allowing it to stand for a day or so, and remixing thoroughly.

A rod of brickwork requires 75 cubic ft. of mortar. "Gauged stuff" is

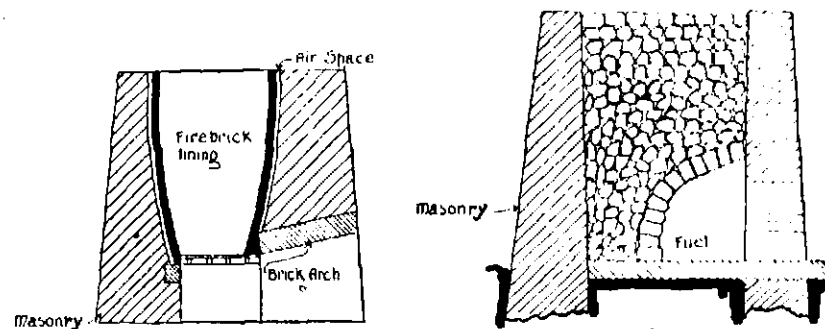


FIG. 210.

FIG. 211.

4-1 plaster of Paris and lime. Plaster of Paris is simply ground gypsum, afterwards calcined. It weighs about 50 lbs. per cubic foot. Alabaster is a form of gypsum of a translucent variety; when of a darker shade it is known as Derbyshire spar. "Coarse stuff" is 1-1 sand and slaked lime with 1 lb. of ox hair to every 2½ lbs. of stuff, while "fine stuff" is pure slaked lime of a consistency of stiff cream. All stuff in plastering is liable to blister, the cause of which is the presence of hard over-burnt particles of lime. The best way of preventing it occurring is by making all such stuff three weeks before use and allowing it to cool. When the sand contains salt, which it should not do, it causes efflorescence, that is, a white furry substance appears on the walls. Saline sand should therefore be avoided, but it is not always possible to tell, *prima facie*, if the sand contains salt. The taste is usually indicative, but we may test for the presence of sand as follows. Put some sand in a glass of water and add nitrate of silver; the presence of salt will cause milkiness, due to the formation of chloride of silver. Outside plastering, usually termed rendering, is done with a first coat of cement and sand 2-1, and a finishing one of 1-1. The surface must be well scored over and wetted, and all joints raked out to form a key for the rendering. One bushel of neat cement will cover about 2½ sq. yards ½ in. thick; if mixed 1-1 of sand and cement, it will, of course, cover double the area, and so on. Rough cast is sand or gravel

mixed with cement or "hot" hydraulic lime. It is laid on a layer of coarse stuff while the latter is still wet.

To sum up our remarks on lime, we may say generally that fat lime is obtained from mountain limestone or pure white "upper" chalk. Slightly hydraulic lime is found at Dorking, Lewes, etc., in Sussex, and is another term for stone lime, referred to before; the latter does not, however, set under water, but is used on most outside building operations.

It will increase in bulk on slaking as much as 75 of its own volume. The actual chemical change which limestone undergoes on burning is that the carbon dioxide (CO_2) is driven off, and also the water, the result of which is the quicklime, also called caustic lime. When water is added, the slaking takes place. Pure lime left in the open air is liable to "air slaking," that is, the CO_2 , which before was driven off is reabsorbed from the atmosphere. Clay is the most important element in lime which produces hydraulicity, hence its use in the manufacture of artificial cements. It will modify the slaking action till, when present as much as 30%, it (the slaking action) is hardly perceptible. Also, silica in mechanical suspension is a useless addition to lime. Clay is soluble silica. The addition of carbonate of magnesia reduces the slaking action also, but will slightly increase the setting power. Metallic oxides also produce hydraulicity, while metallic sulphates are conducive to rapid setting. Lime which is burned, slaked, mixed with clay, and then burned again is known as "twice kilned" lime.

Concrete Blocks.—In making these, only the very best cement is admissible, and all sand should be very carefully screened through a ½-in. sieve and be clean, sharp, and gritty. Very fine sand is not to be recommended. Crushed granite, trap, and limestone are superior to sand. Ordinary blocks may be made in a 3-1 proportion; and when a maximum load on them would not be more than 4½ tons on the square foot, 4-1. Thoroughly efficient mixing is the secret of the whole process; the cement and sand must in all cases be mixed dry and watered through a rose. The "curing" wants particular attention. It must be done in a shed, away from exposed sunshine, but having free access to the winds. The blocks should be frequently sprinkled with water during the process. If possible, allow 1 month for curing; in no case should it be less than 7 days, and this period is too short.

Timbers.—Timber for building purposes is obtained from those trees which are termed "exogens," that is, during growth the tree expands outwards. The sap ascending between the bark and the tree absorbs carbon dioxide and afterwards thickens to firm wood. This circulation is suspended during the winter, and hence we have the well-known annual rings. They are made up of cellular tissue and woody fibre, the strength of the wood along the grain depending upon the tenacity of those tissues, and across the grain by their mutual adhesion to one another. In the centre we have the pith, and outwards come the medullary rays, which, when large and distinct as in oak, are called by carpenters "silver grain." Then there is "heartwood" and "sapwood." In a mature tree we find no pith. Sapwood is known by the "dirty water" appearance it has. Also, contrary to what many suppose, the heartwood is not of necessity, and is hardly ever in fact, in the centre of a tree. It will also be noticed that the rings are thicker on one side of a tree than another. This is the side which, during growth, has had the most sun and air. For practical purposes we divide timber into—

1. *Pine wood* (trees of the coniferous order).
2. All other trees are termed *leaf wood*.

Variety No. 1 contains turpentine. They have a straight fibre, and are specially useful in carpentry which has to bear a direct pull, and for all kinds of planking.

No. 2, however, shrinks unequally in drying because of the great distinction of the rays and rings. They bear shocks and transverse stresses well. Good timber will always show a dark colour, a firm shiny appearance when fresh cut, possesses a good weight, and is free from resin and sap. The detrimental qualities to be looked for are, cup shakes, that is, separation of the annual rings, fig. 212; upsets, caused by injury due to compression, fig. 213; rind galls, the term applied to the formation of new layers over a damaged part.



FIG. 212.



FIG. 213.



FIG. 214.

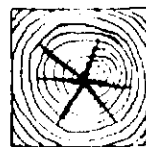


FIG. 215.

Also heart shakes, fig. 214; star shakes, fig. 215; and all sapwood. In building work it is usual to specify that "the timber used by the carpenters and joiners' work throughout to be of the first quality of its particular kind, free from sap, shakes of all kinds, large and loose knots, and all other imperfections, and to be thoroughly well dried and seasoned." Joists, rafters, collars, ceiling joists, and wall plates, etc., are usually made from spruce; framed joiners' work of Archangel red deal; panels of yellow pine; floors of Norway pine or, if subject to much weight or heavy traffic, pitch pine.

The best pine comes from Riga, and the next best from Memel and Dantzic. The wood is principally used for outdoor carpentry and heavy

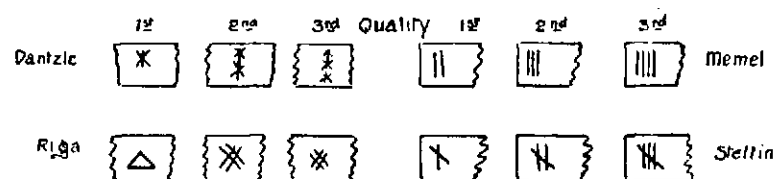


FIG. 216.

work. It is marked as shown in fig. 216. A variety also comes from North America, the latter being used for lighter work. *Larch*, a native timber, is very good for outside work, and is very strong, but it is hard to work and will warp a good deal. *Oak* comes to maturity at about 100 years old, at which time each tree will yield on an average 75 cubic feet of timber. Its colour may be described as a pale brownish-yellow with a shade of green. The annual rings are small. Bad quality of oak can be told by larger rings, which show a dull surface and have a "foxy" hue about them. Oak also has the property of rotting iron nails or other ironwork in contact therewith; it is due to the presence of gallic acid in wood that such action takes place. Oak fastenings are consequently usually made with treenails and hardened pins.

Chestnut, although an inferior timber, bears a resemblance to oak. It should be felled during winter months, and have the bark stripped the year before felling, and should in all cases be "squared" as soon as felled. *Walrus*, seasoning, one week for each inch in thickness is allowed, but when the timber is very thick a little more time may with advantage be given; a log 8 in. thick would hardly season under a period of 10 weeks. All timber used in buildings must be kept free from mortar; slaked lime hastens decay. *Baltic white deal* has well-defined annual rings, usually contains many hard and glassy knots, has a silky lustre when cut fresh, and is subject to streaks of resin in cavities. *Yellow deal* has more of a honey colour, with red-brown knots. It may easily be distinguished by the semitransparent shavings which it yields. *Elm* is a reddish-brown wood, with a light sapwood. Both elm and *beech* are used by engineers for piles, and are very durable when continually kept wet. Norwegian timber is usually very sappy; it is imported directly in the form of planks and match boarding. Pitch pine requires very careful seasoning to prevent shrinkage; it is heavy and has a well-marked strong grain; it is good for work which has to be seen. This grain in the wood is due to the annual rings, not to the medullary rays, which is usually the case. All boards are cut tangent to the annual rings. The Baltic yellow deal referred to above comes from Scotch fir, a timber which is also known by the name of northern pine. What is called Dutch Wainscot is an oak variety. Planks warp so that the concave side produced by the warp was the side of the plank originally nearest the convex side of the tree. In the same way and for the same reason, barks originally square become obtuse-angled, due to the tendency of the stuff to shrink along the lines of the annual rings. To set out the strongest beam out of a log we have the construction shown in fig. 217. The diameter is first divided into 3 equal parts, two perpendiculars are erected, and the rest of the construction is obvious; we may then test the beam for safe load in

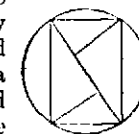


FIG. 217.

cwts. by the familiar formula, $w = \frac{2cbt^2}{L}$. To find the contents of a round log in cub. feet we square the mean $\frac{1}{4}$ girth in inches, multiply by the length in feet, and divide by 144. To those who use the slide rule, the following will be familiar:

C	Length in feet	Cubic feet.
D	12	Quarter Girth. }

Decay in timber.—Both log and balk timber are liable to what is termed dry rot, but sapwood (which, as before stated, is to be avoided) is most liable. It shows itself in a white or grey powder, sometimes in red spots. To avoid it, never use wet unseasoned or sappy timber. Carefully ventilate all ground-floors by perforated bricks. Dry rot will spread rapidly if favoured by warmth, and so in doubtful situations it is advisable to use creosoted joists. There is also a form of preservative now on the market called "Solignum," which seems to have a fairly general use, but unfortunately the author cannot state its qualities from experience. The practice of laying linoleum close up to skirting boards will also favour the ravages of dry rot.

Wet rot may also occur in timber. It is principally found in growing trees or any wood much exposed to rain. *Teak* is a useful wood, which comes principally from Burmah. It is used by engineers, and is very hard, stands well in contact with water, and will resist the ravages of the white ant in

tropical climates. *Green-heart* is principally used in harbour works. It is hard, heavy, tough, and elastic, very inflammable, but will resist the teredo. It is very expensive. All the huge locks on the Manchester Ship Canal were originally made of solid green-heart.

Preserving timber.—Timber requires preserving against the effects of dry rot, wet rot, and in marine work the teredo; perhaps the best known and most efficient process is Bethell's. The process consists of impregnating the timber with creosote oil from which all ammonia has been driven off, the effect of which is to coagulate the albumen in the wood, and so prevent its decomposition, while the bitumen present in the creosote will fill the pores of the wood, which excludes the admission of air and moisture. The timber is first dried and then soaked in hot creosote; more often it is done under pressure, by which the process can only be made really efficient; in fact, the author is firmly of the opinion that simply soaking in the creosote is next to useless, and waste of time and money. By means of suitable pressure in specially constructed cylinders a cubic foot of timber will absorb about 8 lbs. of creosote (or $\frac{1}{8}$ of a gallon). Oak will not absorb more than 6 lbs., however, while red pine will absorb as much as 16 lbs. Creosoting will in all cases tend to render the wood brittle. In modern American practice unseasoned timber is placed in the cylinder, subject to steam at about 100 lbs. pressure per square inch. The action of the steam and subsequent vacuum will remove all sap. Creosote is then run in and the pressure applied as above. The process is much used on railway sleepers. It will easily double their life.

A process known as Bouchière's is to impregnate the wood with a 1 per cent. (or 1 part in 100) solution of CuSO_4 (copper sulphate) and water. This is then run into each log under a pressure of about 20 lbs. per square inch at the end of each log.

Kyanising.—This is a process of preserving timber originally introduced by a Mr Kyan, and so bears his name. The specific used is perchloride of mercury, which must always be stored in a wooden tank. The proportions to use are about 10 gallons of water to 1 lb. of the corrosive sublimate, which quantity will suffice for the preserving of about 45 cubic feet of timber. Large timber is soaked for a period of about 21 days, but for smaller logs a shorter time will naturally suffice. Tepid water must be used to dissolve the crystals efficiently.

Burnettising consists of immersing the timber in a solution of zinc chloride. The proportions are 1 lb. of zinc chloride to 4 lbs. of water, and the period of immersion may be set down as 2 days for each inch of log in thickness. The timber is afterwards dried for a period which may be anything between 14 and 90 days. It is also a useful way of treating ropes and canvas. A 6-1 solution, however, is strong enough for this purpose.

The *Payne process* has the property of rendering wood practically incombustible. The wood is impregnated while in a vacuum with strong sulphate of iron, and sometimes with the addition of sulphate of barium. It afterwards goes under saturation with sulphate of lime, or any other alkaline carbonate, which so renders the iron oxide insoluble. Unfortunately this process sets up slow decomposition and destruction of the woody fibre, which has prevented its general adoption.

To conclude our remarks on timber, it may not be out of place in this article to state what a standard of timber is. It is the recognised way of stating a certain quantity of timber used by timber merchants. The two principal

standards in use are the London and St Petersburg, and comparing them we have—

		Supl. Ft.	Cubic Ft.
St Petersburg	= 120 pieces $12' \times 11'' \times 1\frac{1}{2}''$	= 1320	= 165
London	= 120 „ $12' \times 9'' \times 3''$	= 1080	= 270

—and as commonly used in Ireland, the Dublin standard, which is the same as the London standard.

Asphalt (*vide also Roads*) is a combination, either natural or artificial, of bitumen, limestone, or other calcareous matter. The natural asphalt has two varieties, lake asphalt and rock asphalt. The lake variety is found in and around Central America, notably in the great Pitch Lake in Trinidad. It requires refining, which is done at a temperature of about 400° Fahr., by which means water is drawn off. We then have what is termed "refined asphalt," which contains about 60 per cent. of pure bitumen. The two component minerals are petroleum (a yellow oil), and a hard, black, brittle substance known as asphaltene. It is the latter which predominates in the rock asphalt, and so makes it too brittle to use by itself, hence we have to add mineral oil while the asphalt is liquid. It is then called "asphaltic cement." The natural rock asphalt, which comes from Val de Travers in Switzerland, Seyssel, and elsewhere, is, when *being into it* for use, ground up with a proportion of sand and mixed with pure *tar*, in which state it is termed "mastic." Pure bitumen must in all cases be used, and on no account coal-tar pitch. When making a paving substance, the usual proportions used by municipal engineers and contractors are, asphaltic cement 14 per cent., sand 80 per cent., and calcium carbonate 6 per cent. Crushed granite may with advantage be used in place of part or whole of the sand. Good bitumen is always viscous, and does not lose weight on heating; it must be specified to be free from earthy matter, and be capable of drawing out into fine threads. Artificial asphalt is made from coal-tar residue, and it must only be thinned down by heavy tar oil; 5 per cent. of sulphur added during melting is a great advantage. We very often find spurious imitations of asphalt on the market made from inferior varieties of coal-tar. The consequence is, it is very necessary to submit a doubtful product to some form of test or other. A simple and efficacious way is that recommended by Messrs Greenwell and Elsdon in their treatise upon roads, and is given here *in extenso*:—"Extract with carbon disulphide and filter off the liquid portion. The filtrate is then evaporated to dryness and the residue heated till it is sufficiently dry and hard to grind to fine powder. This powder, about 1 gramme in weight, is treated for 24 hours with 5 cubic centimetres of strong fuming sulphuric acid, and then stirred with twice its volume of water. If coal-tar pitch is present the solution will be of a dark brown or black tint, but natural bitumen will give a light yellow solution." Rock asphalt is chiefly bituminous limestone. The composition of the Val de Travers variety is about 8 per cent. of bitumen, 50 per cent. lime, and 40 per cent. of carbon dioxide. In its natural state it appears brown, but exposure to the atmosphere will cause it to go grey on the surface. Heat will soften it, 180° F. will cause it to disintegrate and fall to powder, while in no case should the percentage of bitumen vary between the limits of 7 per cent. and 12 per cent. Carbon disulphide or ether will dissolve the bitumen, and, if of the pure rock asphalt variety, petroleum will have the same effect, but coal-tar compositions will only give a yellow fluorescence on the application of the latter mineral. For

heavy road traffic it is used in the compressed state, but for footpaths and similar light work, in the mastic state. That for roofing purposes is slightly different both in composition and its mode of application. It is put on in a semi-liquid state, and is in proportion about 12 of asphalt to 2 of grit, and for floors 16-2. Actual asphalt is a mixture theoretically of a number of different hydrocarbons containing oxygen, nitrogen, and sulphur, with more or less mineral matter and ash. What is called "tar concrete" is made from broken stone or gravel of sizes varying from $\frac{1}{4}$ - $2\frac{1}{2}$ ins.; the aggregate should if possible be treated before being incorporated with the tar, but in all cases thoroughly dried. The proportion of tar is usually 12 gals. to each cubic yard of material. Its mode of application belongs to the practice of road-making, and it would be foreign to describe the process in this chapter. Bitumen and sand are often used in sheet form for damp courses in building, and for covering "lean-to" roofs of temporary buildings. Its use in both cases is well adapted to the purpose. Tarred felt is often used in the latter case; it requires tarring and sanding every two years, and in any case its life would not be longer than seven years; and for the reason that no such renewal of a bitumen damp course is possible, it is of paramount importance that when used the very best of its kind be obtained.

We now seem to have touched shortly, and I hope intelligently and to the point, upon the principal materials used in actual everyday building construction. Much more could be said, especially with regard to road materials, which will be fully treated under Roads. To those who, like Oliver Twist, require more, I must refer to various standard works on the subject. But, however, in these enlightened days when steel and iron come so much into the province of the civil engineer and architect, a book of this nature would seem to be incomplete without a few remarks on those valuable materials, and I shall, before concluding, try to put before the reader as concisely as possible the various properties of cast iron, wrought iron, and steel, concluding with a few cursory remarks on other metals in everyday use.

Iron.—Iron, in its various manufactured forms of cast iron, wrought iron, and steel, is by far the widest used metal in everyday use at the present time. The presence of sulphur in any quantity in iron is detrimental to its quality, as it causes what is termed "red shortness," that is, the iron when at a red heat becomes brittle and will not weld. Phosphorus, when present in excess, has the property of producing "cold shortness," that is, the iron which is subject to this becomes brittle when cold, and is practically useless. Hence the necessity of using fuel in the manufacture of iron which is naturally free from sulphur, such as charcoal; and it is hardly necessary to point out the value of charcoal iron, such as that produced in Sweden.

Of *pig iron*, which is the crude product of the blast furnace, there are two chief varieties, grey iron and white iron. Grey iron is produced at a high temperature, and the white at a lower one, and the latter divides itself into two forms, that is, granular and crystalline. White iron is capable of conversion into grey iron by fusion and slow setting, while the process may be reversed by fusion and sudden quenching. No. 2 or No. 3 grey iron is usually specified for castings. Such castings will shrink as much as $\frac{1}{8}$ in. to a foot on cooling, and so patterns have to be designed accordingly to allow for this.

Cast iron is cheap, useful, and easily machined. Its use in building is, however, principally confined to small stanchions and columns, and sometimes joists, but its use is now practically superseded by steel in some form or

other in structural work. It is, however, largely used in smaller items, such as w.c. cisterns, water mains, sometimes for house drains, grids, and a variety of other purposes. The most common defect is blowholes, which are caused by the escape of steam in the mould when made of wet sand. Cast iron contains much carbon, either chemically combined or mechanically mixed, as graphite. Large quantities of silicon cause iron to be hard and brittle; its presence tends to convert combined carbon into graphite, while sulphur has the directly opposite effect. Phosphorus causes the metal to flow well, but excess of either sulphur or phosphorus is dangerous. Manganese has a hardening action and tends to eliminate an excess of sulphur. What are called malleable castings are produced by heating a casting for some days at a red heat in the presence of animal charcoal or red hematite, whereby much of the carbon is driven off. Their use is, however, not usual in constructive work. When examining castings, tap the edges with a light hammer, which should make an impression but not chip the edges off.

Galvanising is a process practised on castings such as cisterns, manhole covers, etc., to prevent decay. It is very efficient as long as the surface produced remains intact. As soon, however, as it gets chipped or otherwise damaged, a galvanic action is set up by the iron rusting, and it speedily spreads and destroys the casting. The process consists of first subjecting the casting to "pickling" in a 1% solution of sulphuric acid, then heating and immersing in zinc chloride, and finally plunging into molten zinc. The article is then beaten while hot to remove an excess of zinc.

A good average specification for cast iron is as follows:—

Two bars, 15" long and $1\frac{1}{4}$ " diameter, should be cast together in one mould from the iron used which should be from the second melting. They must have a taper of $\frac{1}{8}$ ", and be cast in greensand. Two sets of bars must be cast, one of the first and one of the last of the metal going into the casting. These bars, when over a span of 4' 6", must bear a central load of 500 lbs. The metal should be good, soft, grey, cold blast iron, sound and clean, cast from the second melting, free from injurious cold shuts, blowholes, etc., true to pattern and of workmanlike finish. All sharp corners to be well rounded, and triangular fillets cast at all internal angles.

Wrought iron may be specified so:—

Wrought Iron.—The wrought iron shall be "best best" iron, of tough quality, with smooth skin, free from lamination, and all other defects, and of full scantling specified, capable of bearing tensile stress of 22 tons per sq. in. of section along the grain, and shall extend at least 8% of its length before fracture.

Bolts shall be made to proper lengths and sizes, shall have square heads and nuts, and screwed with full Whitworth standard threads, head being in one with the shanks, and not less in thickness than the diameter of the bolt, and proper washers shall be put under the heads and nuts.

Hexagon head bolts would of course be specified according to circumstances.

For soil pipes, water mains, and other cast-iron work in which galvanising or painting would be unsuitable, a coating of Dr Angus Smith's patent solution is used. A process known as the "Barff" process consists in heating the iron to redness and subjecting it to superheated steam at a high pressure, which forms a protective coating of magnetic oxide on the surface.

Steel.—In these days, by means of the perfected Bessemer and open-hearth processes, steel is so cheaply produced that it has entirely superseded the use of wrought iron and to great extent cast iron in constructional work. It is stronger, and hence for given loads the sections may be lighter. Professor

ry Adan.

contain, south of this "slip" is nicely adjusted to suit the fireclay body, so that the slip is vitrified by the intense heat in the kiln, and forms a smooth, glassy surface inside, thoroughly incorporated with the body of the pipe, the fireclay body is sufficiently "fired," and ready for the salting process already described. This process produces a strong pipe, smooth inside, non-porous, and deviable, not deteriorated by sewage or the test of time, and greatly superior to the common salt-glazed pipes.

Stoneware pipes are frequently used in England, but the great defect of these is their brittleness, which renders cutting, in the hands of an ordinary workman, almost impossible. The stoneware is harder and closer grained than fireclay, and consequently less porous; but, owing to the drawback mentioned above, they do not appear to be much in favour where the best fireclay pipes are available.

The sockets are usually too small for efficient jointing in the smaller sizes, and they are only made in 2-ft. lengths as a rule, which means an undesirable multiplication of joints in a drain. The price is higher than for fireclay pipes of the best quality.

The following is the analysis of the fireclay from which fireclay pipes are made:—silica, 57.35; alumina, 38.60; oxide of iron, 1.85; lime, trace; magnesia, 1.22; potash, 1.44; soda, 0.96,—101.49.

It is obtained from the coal measures, and is admirably adapted for the manufacture of drain pipes.

We now appear to have touched lightly on all the usual building materials, and I cannot but impress on all engaged upon works of construction the great importance of a knowledge of the essential properties of the materials employed, of how to specify the best material, and how to detect the bad. More especially do I advise the student of civil engineering to carefully study the subject, as, besides being useful in everyday work, it forms a fair percentage of the examinations at the present day.

CHAPTER XIV.

FOUNDATION AND PILE-DRIVING.

Soils.—A weak soil having a uniform texture is always more reliable than a stronger soil of unequal density.

In ordinary foundations we have three main divisions, viz.—

- i. *Rock Foundations.*
- ii. *Foundations on firm earth*—(e.g. sand, gravel, and hard clay).
- iii. *Foundations on soft earth.*

✓ The main points in designing foundations are—

1. To get the base perpendicular to the pressure.
2. To make the area of the base large enough to withstand that pressure.
3. To make sure that the centre of resistance does not deviate beyond certain limits from the centre of gravity of the masonry in question (see Retaining-Walls).

Rock foundations are simple enough. It is only necessary to cut away the loose rock which may have become decayed, to dress the hard rock to either a plane or stepped surface according to circumstances, and finally to make good the places, if any, which had to be cut away. This would be done with concrete or, if necessary, solid masonry of the rubble type. Ordinary good rock may be considered as strong as good red brick, at least for resisting pressure, viz. about 160,000 lbs. per sq. ft. But it is not usual to load it to more than $\frac{1}{3}$ of this, say 20,000 lbs.; while if the rock is sandstone of a soft quality, 4500 lbs. is ample. The fundamental equation in reference to foundations of all classes is that enumerated by Professor Rankine.

We have x = depth of foundation in feet.

w = weight of 1 cub. ft. of the earth (or rock).

ϕ = its natural angle of repose (Chapter VI.).

Then if we have a load (say that of a building, bridge, pier, or engine foundation *) = P_0 per sq. ft. in lbs.,

we have
$$\frac{wx}{P_0} > \left(\frac{1 - \sin \phi}{1 + \sin \phi} \right)^2 \quad (188)$$

which means to say that the sign $>$ signifies that the first term in the equation must be at least equal to the second or greater than it (never less),

* Allowances for vibration would be made.

Henry Adams defines steel as "all those malleable forms of commercial iron which contain iron and carbon produced by a state of fusion into a malleable ingot," all the ^{work} being either wrought or cast iron. In the Bessemer process molten iron from the blast furnace is conveyed to a converted pear-shaped vessel similar to fig. 218. It is then subjected to an air blast for about ten minutes. The air enters by the hollow trunnions and comes up through the bottom. There is a copious evolution of sparks, and most of the carbon is driven off and only pure wrought iron remains; a known quantity of carbon is added in the form of Spiegeleisen, which is a compound of iron with about

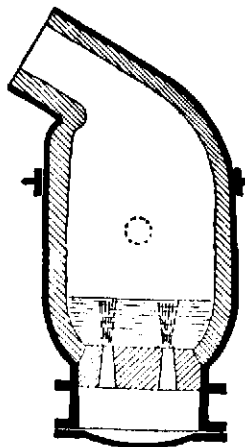


FIG. 218.—Bessemer Converter.

20% of manganese and 5% of carbon. If on rolled steel joists there is no distinguishing mark, foreign joists may be distinguished by the unusually square edges and frequent bastard sizes, but most reliable makers stamp their name on the joists, and it is compulsory that all foreign joists should be marked with the name of the country where they were rolled, such as "Made in Belgium." Steel in construction will safely stand stresses of $6\frac{1}{2}$ tons per sq. in.

Rivet steel should be of the best quality of open hearth steel of an extra soft grade, of 45,000 to 50,000 lbs. ultimate strength, with a maximum of phosphorus of .06% and of sulphur .04%. A bar should be capable of bending cold 180° flat on itself without fracture on the outside. On breaking off it should show a close-grained silky fracture, and all rivets which show a dark colour should be rejected.

Lead is a metal employed to a considerable extent by builders. It melts at 400° F., and has the property of becoming coated with a film of oxide which

will protect it from further corrosion. It is made in sheets of various thicknesses, and usually stated by its weight per square foot in lbs. Thus for roofs and main gutters 7-8 lbs. lead would be used; tips, ridges, and small gutters 6-7 lbs., flashings 5 lbs., while for soil-pipes 10 lbs. A good basis for calculation is the fact that for every $\frac{1}{16}$ of an inch thick, lead weighs 1 lb. per sq. ft.

Zinc melts at 700° F., and has the same protective film as lead. It will not, however, stand sea air or the acid-laden air of manufacturing towns.

Tin, which is not used in the pure state in practice, is a soft, ductile metal, which melts at about 460° F. Clean sheet iron coated by immersion in molten tin is called tin plate.

Copper has the property of rapidly losing strength at 600° F. It is used in very expensive work for roofs, and acquires a protective film of carbonate of copper, commonly but wrongly called verdigris. Verdigris is really a compound of copper with acetic acid, known as acetate of copper.

Bronze is a mixture of copper and tin.

In Bell Metal, which is also a mixture of these two metals, the proportions are 6-1

while in ordinary gun-metal the proportions are 9-1

Brass is an alloy of copper and zinc, and is in most cases weaker than bronze, but, on the other hand, more fusible and ductile

The usual proportions are—

Ordinary brass 2-1 copper and zinc } by proportion.
Malleable brass 4-1 " " }

Sewer and Drain Pipes.

In drain pipes there are great differences in quality, and so it is a matter of doubt whether the pipes laid are actually of the class specified, and for this reason it would certainly be useful to engineers, architects, and others if manufacturers stamped their goods according to the class they belong to.

It is not practicable to have a competent clerk of works on all drainage work who can reject inferior pipes before they are laid and jointed, so that the engineer may often see only the outside of the pipes when inspecting the work, and awkward differences of opinion may arise with contractors as to the actual class of the pipes laid.

Many fireclay drain pipes are very unsuitable for house drainage owing to their being bent, cracked, buried inside, untrue in section, making good jointing impossible, defective glazing, porous, insufficient size of sockets for efficient gaskin and cement jointing, no glaze in sockets, etc.

"Common" Scotch "salt glazed" fireclay drain pipes are very good for certain purposes where a uniformly smooth bore and impervious glaze are not of great importance, but it would be quite a mistake to use an inferior class of fireclay pipe for a house drain, especially where it is laid under the building, and this in the majority of town houses is unavoidable.

It must also be remembered that the sanitary authorities of most towns do not permit new drains to be covered in until their surveyor has tested and approved them, and the use of inferior pipes would often lead to failure under the hydraulic test.

The common fireclay pipes have usually a "salt-glaze," which gives a fairly smooth surface in pipes made from ordinary fireclay; but if ironstone exists in the clay, it runs in the firing and makes "burrs" in the pipe.

The "salt-glaze" is made by throwing salt into the kiln and fires when a certain degree of heat is attained.

The class of drain pipe most in use in Ireland and Scotland is the "extra-glazed" fireclay socketed pipe. This is a highly suitable class of pipe for all ordinary drains and sewers of moderate size when made from a good fireclay free from foreign matter, and when well made, straight and true in section, not less than $\frac{3}{4}$ in. thick in 4-in. to 6 in. pipes, sockets at least $2\frac{1}{2}$ in. long and made parallel with the barrel of the pipe, so that the weight of superincumbent earth may not fall on the outer edge of the socket, as in cases where the sockets are tapered. The latter defect becomes more important when the drain is laid on a concrete bed, which is the practice in good work, and if the pipes are not "cuddled" with concrete between the sockets to distribute the weight evenly.

These pipes are generally made in 3-ft. lengths, but are also supplied in 2-ft. and 2-ft. 6-in. lengths, all made true in form, and can be easily cut by a skilled labourer.

Fireclay sewer pipes are made from the finely ground and tempered fireclay, by being pressed by steam-driven revolving "knives" through a cast-iron cylinder, to which is attached a polished cast-iron "die," suitable to the size of pipe wanted for the time being.

The pipes, when sufficiently "stiff" to handle, are finished, and the "slip" glaze (which is composed of fine, dense, vitreous clay, with a mixture of flint glass and other ingredients) is poured through them, thereby giving them an internal coating of the mixture.

and this equation is mostly used in reference to soft earths or those in which there is a liability to slip owing to the presence of water, the formula being hardly necessary with rock or compact earths, because the natural practical depth of the foundation is almost sure to be greater than x , being about 3 ft. for sand and 4 ft. for clay, in this country at all events. In America and in cold climates generally we may go further to avoid the effects of severe frost. The same remarks regarding the diversion of surface water as were explained in Chapter VI. hold good, and the use of catchwater drains. In firm earth the usual safe intensity of pressure would be between 2500 and 3500 lbs. per sq. ft. All trenches for foundations must be carefully drained before putting them in; and if soft ground, we could make it deeper and fill up with more stable earth.

In this case we have w = weight per ft. cube of the filling.

w_1 = " " " earth.
 ϕ = angle of repose of the former.
 ϕ_1 = " " " latter.

Then work out the values $\left(\frac{1 - \sin \phi}{1 + \sin \phi}\right) = K$ and K_1 for each of the two materials,

and we have the following equation, also giving the depth of the proposed foundation *before* the filling is put in. The value is

$$x_1 = \frac{P_1 K_1^2}{w_1 - w K^2} \quad (189)$$

P_1 being the greatest intensity of pressure expected. The filling must be well rammed.

When such filling is concrete we may load it to 6500 lbs. per sq. ft. But it must be well set at not weaker than 6 to 1.

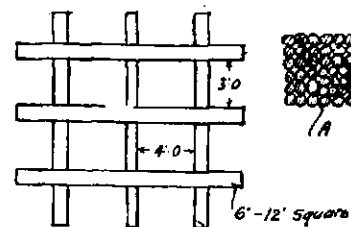


FIG. 219.

The class of foundation which most frequently will present difficulty to the engineer are submerged foundations. The immersed parts are then taken to be less in weight = 62.5 lbs. per cub. ft., this being the weight of a cubic foot of water in practical calculations. In very wet and soft ground we employ timber platforms as shown in fig. 219. We could also place the same foundation on round piles as shown at A. The piles are 6"-7" in diameter, and always driven in towards the centre from the outside. This is so as the centre pile will form a sort of wedge. Long piles may go into firm ground. When so placed they will safely bear 1000 lbs. per sq. in. of surface. But if they are in soft ground, where they stand by friction alone, 200 lbs. is ample. Straight-grained elm is the best. All the bark must be stripped off before use. The ordinary form of pile driver used by engineers is called the monkey engine, and it is shown in fig. 220. It is worked by hand

power. Large ones have a steam engine, usually of the travelling-crane type. For application to such an engine we have the following formulae—

W = weight of ram (lbs.).

H = its fall (in ft.).

w = weight of pile (lbs.).

P = length of pile (ft.).

A = sectional area of pile (sq. ft.).

E = modulus of elasticity of the pile (*vide* Concrete Steel).

D = set of pile by the last blow (ft.).

L = greatest load on the pile (crushing).

l = " " (safe).*

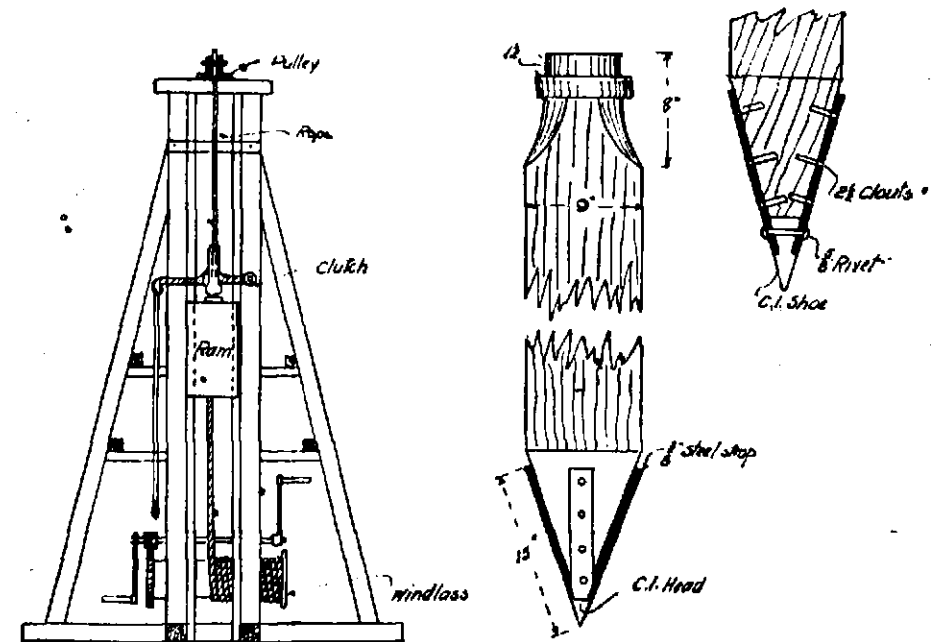


FIG. 220.—Pile Driver.

FIGS. 221 and 222.

Then
$$L = \sqrt{\left(\frac{4EAWH}{P}\right) + \left(\frac{4E^2A^2D^2}{P^2}\right)} - \left(\frac{2AED}{P}\right) \quad (190)$$

Again, if A = sect. area of pile in sq. in.,

$$W = w \left(\frac{wH}{5AP} - 1 \right) \quad (191)$$

A short fall with a heavy ram has more effect than a long fall and a light one, the number of ft.-lbs. of energy in all cases being the same.

In the *Transactions of the Institution of Civil Engineers*† we have the following conclusions, viz. :—

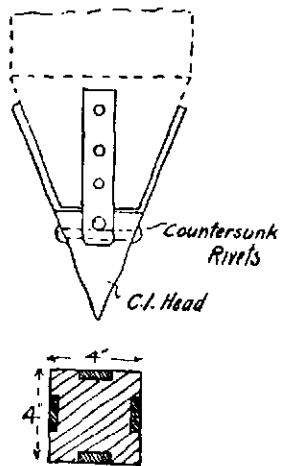
1. More waste of labour is caused by blows than by constant, steady pressure.

* Usually = $\frac{1}{2} L$ or $\frac{1}{4} L$ in specially good places.

† Vol. xlv.

2. One blow exercising the same mechanical power as that of a steady known pressure will not produce the same effect as that pressure.

3. The mechanical power necessary to effect temporary alteration of substance up to its limit of elasticity, if applied through the medium of blows, will not affect that substance up to its limit of elasticity. When a pile will not go more than $\frac{1}{2}$ of an inch ($=D$) for every 120,000 ft. lbs. applied to it,



FIGS. 223 and 224.—Detail of Shoe.

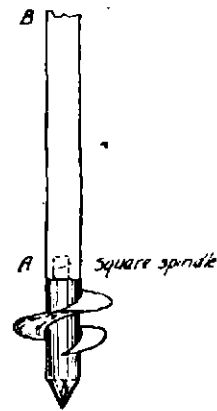


FIG. 225.—Screw Pile.

we can safely say it is driven home. When they are so driven in soft ground, the surface for about 3 ft. deep is excavated and filled with concrete. The capping is the same as in fig. 219, only the string-pieces and cross-pieces are halved (see Carpentry) into each other, and treenails used for fastening.

In figs. 221–224 are shown forms of piles in practice. They are self-explanatory. Green heart piles do not require any shoes. In addition, we have the screw pile, an invention of a Mr Mitchell. It is used in deep water

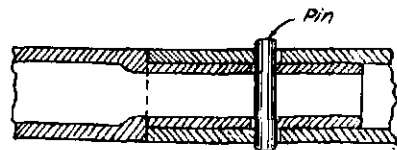


FIG. 226.

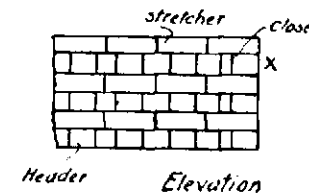
for foundations of piers and jetties, lighthouses, etc. It is shown in fig. 225. It has a large spiral iron flange, having a square spindle at A, and is screwed down to any requisite depth, the rod AB being in screwed sections which can be added as the pile goes down. A better form of joint appears in fig. 226. It may be screwed down by means of an ordinary capstan and levers, especially in foreign parts where bullocks are employed, but in Britain it is usual to use ropes attached to a crab winch, steam or hand driven. Another form of pile is that in which the shaft is hollow, to allow water to be forced through it. The purpose of the water is to remove the sand below the foot

CHAPTER XV.

BRICKWORK AND MASONRY.

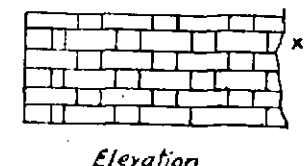
ALTHOUGH the subject of brickwork and masonry belongs more to elementary treatises on Building Construction than to a book of this class, yet it would seem to be incomplete unless a few words were said on the more important points of the subject, especially as to the heavier classes of masonry which fall into the hands of the engineer, such as dock walls, etc.

Brickwork.—As probably the reader is aware, brickwork is divided into two classes of what are termed "bond," that is, English and Flemish. There



Plan on Course X
English Bond

FIGS. 243 and 244.



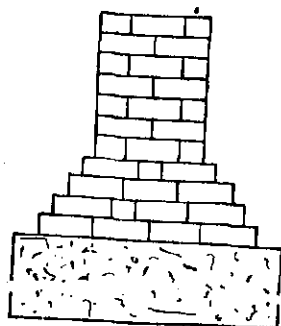
Plan on Course X
Flemish Bond

FIGS. 245 and 246.

are others, all more or less of a fancy nature. For general strength and stability in structures, nothing can compare with the English bond, of which an example is given in figs. 243 and 244. Here we have alternate courses of *headers* and *stretchers*, with what we call a *closer* in every alternate course to get the bond. The average brick (their composition was described under Materials of Construction) measures $9'' \times 4\frac{1}{2}'' \times 3''$, a size which includes one mortar joint. They are laid in *courses* 3 in. deep. The joints between the courses are the *bed joints*, the corner bricks are the *quoins*. The vertical joints are the *perpenals*. Bricks laid longwise in the wall are called *stretchers*, those crosswise *headers*. Pieces of bricks are called *bats*. The horizontal

distance between the vertical joints (or perpend) of two successive courses is called *the lap*. It should equal $2\frac{1}{4}$ in.

Figs. 245 and 246 show the Flemish bond. Its use is for appearance only. It is used in buildings and garden walls, not usually in engineering structures. The bottom of walls, spread out to comply with the conditions



Footings

FIG. 247.

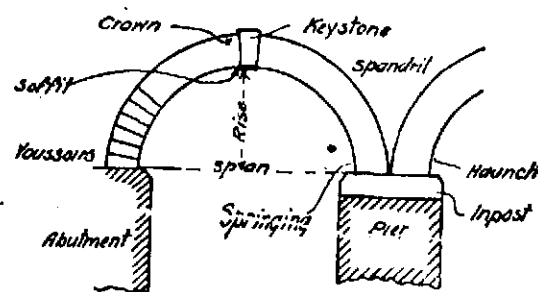


FIG. 248.—Arch and Technical Terms.

of the previous chapter, are called the *footings*. Bricks in footings must be laid as headers as much as possible; where stretchers are necessary they should be in the centre. Fig. 247 gives an example of a properly designed footing, the bottom width being twice that of the wall. The set-off on each side of



Flat or Flush Joint

FIG. 249.



Jointed Flat Joint

FIG. 250.



Keyed Joint

FIG. 251.



Struck Joint Overhung

FIG. 252.



Struck Joint

FIG. 253.



Open Joint

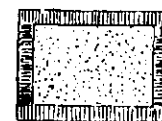
FIG. 254.

Flushed Joint

each course should be $1\frac{1}{4}$ in. ($=\frac{1}{4}$ brick total). When such set-offs occur in any other part of the wall, on either side, forming a ledge, that ledge is called a *corbel*; their use is to support various parts of a structure. Regarding brick arches, fig. 248 gives a diagram in which all the technical terms are explained. Figs. 249-253 give examples of joints in brickwork.

Masonry.—The foregoing terms apply also to masonry. In masonry we often have stones going from back to front of the wall; they are called *through stones*. Their advantage is questionable; they may taper in width, but never in depth. All bed surfaces in masonry must be worked to plane

surfaces. It is the duty of the engineer to see this is done. Unscrupulous masons try to make them hollow and get a better appearance on the face this, of course, is not "engineering" at all, and a very bad practice. The joints so formed are called **Flushed Joints**; they are liable to splinter by the load, see fig. 254. When stones have their faces brought to a fair surface they are said to be *dressed*. The projecting courses often seen at the base of a wall are called *plinths*. When stones are sawed we have a surface left by the saw; this is called *half sawing*. The stones on the face of a wall or of



Pointed

FIG. 255.



Vermiculated

FIG. 256.



Hammer Dressed

FIG. 257.

piece of masonry such as a bridge are brought to various kinds of ornamental facings. Of these, some examples occur in figs. 255, 256, and 257, of which perhaps the hammer-dressed has the best effect. Coming to stone walls, we have three main divisions.

1. Rubble for various purposes when the work would not be seen, such as backing retaining-walls, rough boundary-walls, pitching, and a variety of works which will from time to time be mentioned (*vide* Walls, Weirs, Bridges

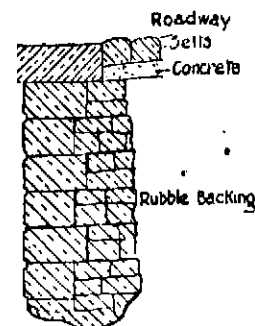


FIG. 258.

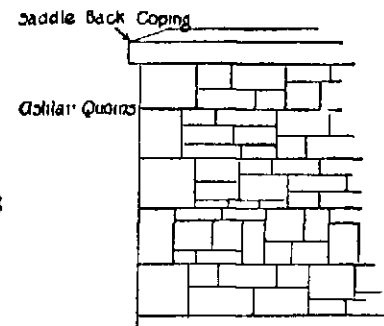


FIG. 259.

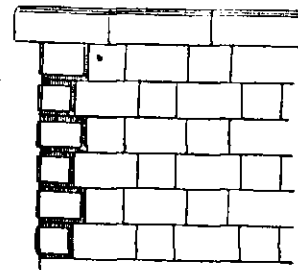


FIG. 260.

etc.); it occurs as random rubble, fig. 258. Dry rubble, uncoursed rubble in mortar, random rubble built in courses, squared rubble uncoursed and coursed figs. 259 and 260, show squared coursed rubble. Then we have *ashlar*. These are stones carefully worked, usually over 12 in. deep; the joints would not be more than $\frac{1}{4}$ in. It is used for facing only and quoins, fig. 259, and is in heavy work invariably backed with rubble (fig. 258) or brickwork. Block in course much used by engineers in quay walls, river walls, etc., is shown in fig. 261. It is a heavier form of squared coursed rubble.

Coming to the joints used in engineering masonry, we have the joggle fig. 262. It is used in landings to prevent movement between the stones joined, and so retain a level surface between them. The table joint appears in fig. 266. It is used in sea-wall work, and its object is to prevent lateral

motion of stones, but it is very expensive, and that is why engineers now are so much in favour of casting large concrete blocks on the site for such work, with table joints or other suitable forms in them. Then we may have what is called a cement joggle. It consists of a "vee" joint on two adjacent stones.

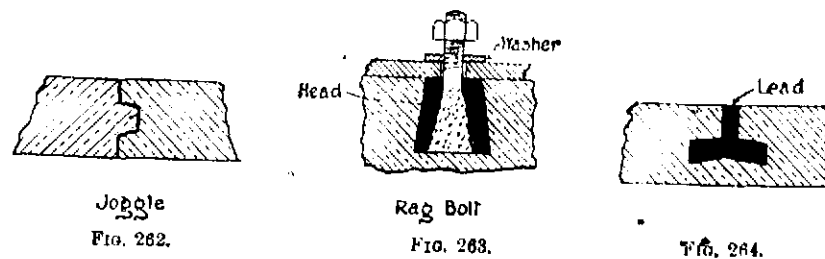


Block in Course
FIG. 261.

The space they form when together is run up with cement, 2 to 1 or neat. Metal cramps are much used to tie the copings of sea-walls together; fig. 265 shows one. The cramps, which are made of thin bars of metal of a size and sectional area according to the work, are turned at the ends downwards. These ends are made rough. Mortises are then cut for them of dovetail shape and the cramp is run up with lead; they are made of iron often. It is of course liable to rust, and sometimes, though not often, bronze is used. They can, if preferable, be run up with cement to prevent the galvanic action that lead is known to set up.

Sulphur and sand and asphalt have also been tried.

Fig. 264 shows another way of effecting the same ends as the cramp. The mortises must be dovetailed well and the lead afterwards caulked.



Joggle
FIG. 262.

Rag Bolt
FIG. 263.

FIG. 264.

fig. 267 we have another joint called the bed dowel. The dowel is made of slate. For fixing things to masonry, such as bolting engines down, etc., and railings, the rag bolt is used, shown in fig. 263. In large machinery, however, special bolts, grouted up in cement, are used. They are attached to plates at the bottom, which are firmly secured to the masonry. For lifting masonry various forms of apparatus are used. We shall consider lifting machinery in a later chapter, but a very useful device much used for the purpose of attaching blocks of stone to the crane hook is called the Lewis. It is made as shown in fig. 268.

In the form shown in fig. 268 the two end pieces are inserted, then the centre one to wedge them up, and finally the ring, pin, and taper key. Fig. 269 shows a simpler form. Fig. 270 shows two ways of breaking joint in finished masonry. In no case should the headers form less than $\frac{1}{4}$ the total superficial area; $\frac{1}{3}$ is usual. Block in course with stones 7-9 in. deep is much used in spandrels, wing walls, and the faces of retaining-walls. In coursed rubble masonry the courses should be 12 in. deep if possible; the breadth of the headers should be $1\frac{1}{2} D = 18$ in., and tail into the wall 3-5 times D . But the stones placed between them may be smaller, say 2 to the depth of each

course (fig. 271). A cubic yard of such material requires about $1\frac{1}{2}$ cub. yds. of stones and $\frac{1}{4}$ cub. yd. of mortar. When ashlar and block in course are backed with rubble masonry, a very prevalent practice in engineering, the tails of the bondstones (or headers) may be quite rough, and may taper sideways, but *never* in depth, as stated. The rubble must be carried up at the same time as the rest of the wall, and have courses the same depth as the face. For setting out walls with a batter face, batter rules are used, simply a plumb rule, with one side cut to give the required batter when the bob hangs

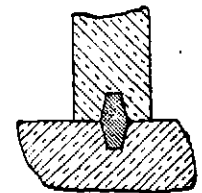
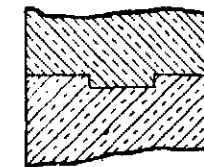
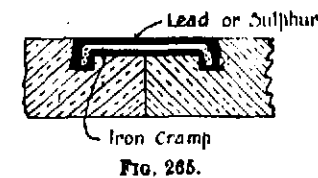
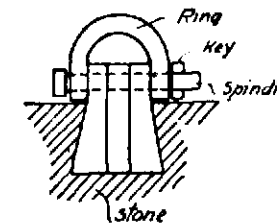


FIG. 266.

FIG. 267.

vertically, but curved walls require a "face mould." They must be most accurately set out; each course of stones must be marked upon it. A plumb bob is provided. Curved work should, however, be avoided as much as possible. A rood of masonry is 36 sq. yds., — feet thick being stated, but cubic yards is the usual engineering measurement.

Regarding masonry and brickwork pillars, the crushing weight of stone



Lewis
FIG. 268.

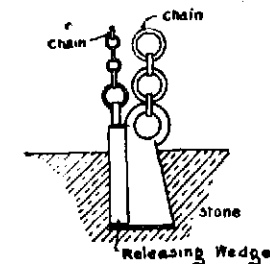


FIG. 269.

is considerably more than would cause fracture only, being often nearly double. In practice, pillars of brick or stone should not be higher than 12 times the least thickness at the base (exclusive of footings). After this they lose strength rapidly; in fact, at a height of 24 ft. we reduce the strength $\frac{1}{10}$ ths, at 30 times $\frac{1}{2}$, and at 40 to $\frac{1}{4}$. A factor of safety of 5 should be employed in such work if a dead load; if the load is live, 10 is the factor.

For best stock brick in Portland cement, engineers usually adopt a safe load of 5 tons per sq. ft. If laid in best lias lime mortar, 3 tons per sq. ft. The joints should be flushed up. Such expedients as lead joints have been tried. They appear to have been a failure.

All stones absorb water to a more or less degree. For such reason they

(like bricks) are always wetted before laying on mortar, to prevent too much moisture being absorbed from the mortar.

The average absorption is as follows:—

Granite	$\frac{1}{200}$ to $\frac{1}{100}$	of its volume.
Limestone compact	$\frac{1}{100}$ to $\frac{1}{200}$	"
Kentish rag	$\frac{1}{70}$	"
Compact sandstone	$\frac{1}{10}$ to $\frac{1}{20}$	"
Portland stone	$\frac{1}{8}$	"
Bath stone	$\frac{1}{8}$	"

An important branch of the study of masonry and brickwork is that of taking out quantities for the preparation of bills. The work is such as is not readily grasped, and wants much experience. It is also somewhat tedious to the beginner, who, however, may save much worry by the intelligent use of a slide rule. However, like everything, "every man to his trade" (or profession), and the particular class of work is usually placed in the hands of quantity surveyors, as they are called, who, by reason of their specialised work, have many labour-saving devices and tricks. The first of being able to take off quantities is a great help to the engineer, but space does not permit of the subject being treated here. We refer the reader to standard

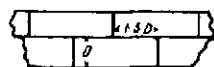


FIG. 270.

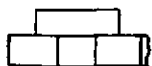


FIG. 271.

text-books.* At the same time it may not be out of place to give an example of a small bill, say, for example, that of a sewer manhole, likely to be quite a common occurrence to the reader, and will give him an idea of how such bills are usually made.

In many modern bills of quantities for sewerage work † the ordinary manholes are not billed in detail, as the depth of each manhole will vary from the others, and the cost to the client by reason of the amount of space occupied by such details in the lithographed or printed bills is not commensurate with the advantages to be derived therefrom; and further, contractors who are accustomed to such work can closely estimate its value merely by an examination of the detail drawings of the manholes. The item would be billed in the quantities as "No. 1 manhole, 12 ft. deep, similar to standard detail drawing." Another method is to take off and bill separately, as an appendix, the quantities of (a) the manhole shown on the standard detail drawing, (b) the quantities for addition or deduction for variation of each 1 ft. in depth, and (c) schedule for variations in invert. In this case the item would be billed as—

No. 1 manhole, 12 ft. deep, as per detail quantities annexed.

Add for invert and benching, being curved on plan E.O.

Add/ Deduct for invert, being 9 in. diameter E.O.

Each manhole should be numbered on the plans and sections, and should be referred to by that number in the quantities.

The method of billing the detail quantities would be as follows: Standard drawing No. —. Manhole 10 ft. deep, to invert on straight 12-in. sewer. Memo.—The lengths of pipe sewers will be measured through manholes. the excavation for which is taken E.O. sewer trench.

* Quantity Surveying, by Leaning.

† Building World.

Quantity.	Description.	Rate.	£	s.	d.
— yd. cube	Excavation to required depth, including all timbering and pumping				
— yd. cube	Fill in and ram excavated material to back of manhole walls				
— yd. cube	Remove all surplus excavated material and provide tip				
— yd. cube	Cement concrete (6 to 1) in foundations, laid to falls and curves				
— yd. sup.	Brown brindled brickwork in cement reduced to 3 in. in walls				
— yd. sup.	Ditto in arch with radiated bricks				
— yd. sup.	Facing walls with best pressed Staffordshire blue bricks E.O.				
— yd. sup.	Following inner ring of arch with radiated ditto E.O.				
— yd. sup.	No. 1 York stone springer 3 ft. 6 in. long out of 12 in. by 9 in., worked to detail and building in				
— yd. sup.	Rake out and point joints of all exposed brickwork				
— yd. sup.	Forming 4½-in. invert in one ring of radiated best pressed Staffordshire blue bricks in cement				
— yd. sup.	Forming benching in square ditto				
— ft. lin.	Benching being bull-nosed on edge E.O.				
— ft. cube	No. 2 Cut square and build in ends of pipes with sand joint and two-ring arch in radiated blue bricks				
— ft. cube	No. 8 Foot-irons and building in				
— yd. cube	No. 1 In 9-in. by 3-in. York bed stones tooled on exposed edges and to receive cover				
— yd. cube	Circular cast-iron open cover with 20-in. clear opening, complete with galvanised iron dirt box				
— yd. sup.	Cement concrete (6 to 1) 6 in. thick under paving around cover				
— yd. sup.	4-in. granite cubes laid on bed of ashes or sand and grouted with cement mortar				
Item	Make good surface of roadway around and over manhole, and remove all rubbish				

Total cost of manhole 10 ft. deep on straight 12-in. pipe sewer

£

VARIATION IN DEPTH.

Quantity.	Description.	Rate.	£	s.	d.
— yd. cube	Excavation a.b.				
— yd. cube	Fill in and ram a.b.				
— yd. cube	Remove surplus a.b.				
— yd. sup.	Brown brindled brickwork in cement to walls a.b.				

Total addition or deduction for each 1 ft. variation in depth

£

VARIATION IN INVERT.

Quantity.	Description.	Rate.	£	s.	d.
Add	Invert and benching being curved on plan				
Add	Ditto, being formed to receive a branch junction				
Add/	Deduct invert being 9 in. diameter instead of 12 in.				
Add/	Deduct invert being 15 in. diameter instead of 12 in.				
Add/	Deduct invert being 18 in. diameter instead of 12 in.				

CHAPTER XVI.

WALLS.

1. On the Stability of Walls (*not retaining-walls*).—Take the simplest case of an ordinary wall, as in fig. 272. P = centre of pressure and x = its moment. W = weight of the particular wall per cubic foot, because p = pressure taken over 1 ft. wide of the wall. CG = centre of gravity of the

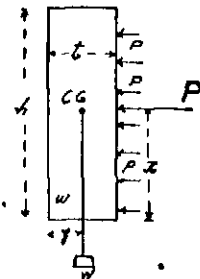


FIG. 272.

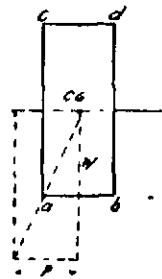


FIG. 273.

wall, while y = the moment of W . In this case P is assumed as a wind pressure only, and is therefore uniformly distributed. In order that such a wall may be in equilibrium, we must fulfil two conditions, that is—

✓ 1. The greatest height shall not be more than h ,

and
$$h = \frac{Wt^2}{P} \quad (195)$$

✓ 2. The least thickness shall not be less than t

$$t = \sqrt{\frac{Ph}{W}} \quad (196)$$

Fig. 273 shows the parallelogram of forces applied to the above reasoning.

Any wall will tend to slide upon its bed joints. This tendency now is directly proportional to the coefficient of friction of the mortar. The letter which signifies its value is μ . In all such calculations we can take safely the average wind pressure at 30–40 lbs. per sq. ft. Fig. 274 and Table A* can now be taken conjointly. Then fig. 275 will show the reaction when the resultant acts through the centre of the wall, fig. 276 when it is away from the centre, and fig. 277 when it acts in the middle third of the wall. This

* End of chapter.

latter position is usually held by engineers to be the correct position for the resultant. But although it may cause *no tension* in the inner joints, yet it may cause *too much* compression on the outer ones, and this maximum pressure is equal to

$$\frac{2wh}{3} \quad (197)$$

Now considering there is no such tension, fig. 279 shows the theory. The base of the parallelogram is equal to three times x ,* and the side of the triangle is twice the depth of the parallelogram. Figs. 280 and 281 are other diagrams based on the same reasoning. Obviously on such walls the wind pressure will vary with the length and height of the wall. If a long wall fails by wind pressure, part *may* stand while part falls down. A reasonable length to take would be one and a half times the height, or fifteen to twenty times the thickness. Such walls are also liable to overturn by frost.†

Now Table B and fig. 283 may be considered conjointly. It will be noted that at 20 the curves for tension and compression cross each other. That proves to us that above 20 ft. in height walls fail by too much compression on the outer edges, and below by too much tension on the inner edges. Table C is compiled on the assumption of a wind pressure (maximum) of 40 lbs. per sq. ft. In reference to the same, we have a special curve in the diagram, fig. 283, marked Hurst's formula. Line A, however, is based on the fact of

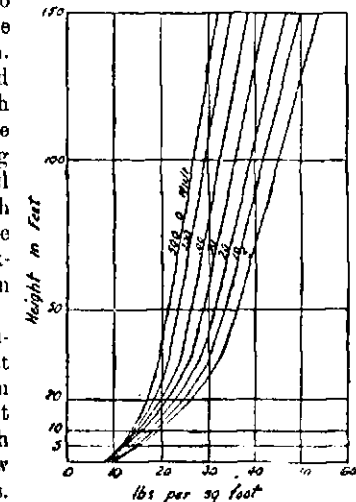


FIG. 274.

$$h = 6\frac{2}{3}t \quad (198) \checkmark$$

and
$$t = \frac{3h}{20} \quad (199) \checkmark$$

while curve X assumes a thickness of one-ninth the height. We said before that we consider lengths of wall of about fifteen to twenty times the thickness; it is at intervals of about this distance that it is usual to place buttresses or counterforts in the walls. A *buttress* is a projection which is on the *leeward* side of the wall, while a *counterfort* is on the *windward*. Fig. 282 will explain them. Panelled walls are, of course, weaker than ordinary ones. They are calculated in the same way, due allowance being made for the reduced weight of them.

By suitably thinning a wall towards the top we can save something like 20 per cent. of material, but it is doubtful in many cases if it is worth while to do so on account of the extra labour. It is the engineer's business to

* $x = \frac{1}{3}t$.

† By reason of the joints opening up.

The length of the arch along the centre line of the face of the arch from springing to springing is multiplied by the length from face to back in order to obtain the superficial area from which the opening for the shaft is deducted. Minor details such as centering, forming opening in arch for shaft, etc., are not usually specifically mentioned. Similar details would be prepared for each type of manhole.

It is also necessary that the student of engineering should have some knowledge of the labour on stonework. When a stone is sawed, the surface left by the saw is known as *half-sawing*, because half the cost is charged to each part of the separate pieces. A stone which comes straight from the quarry is said to be *self-faced*. The term also applies to a stone split in two, but the more irregular angles of a stone are chipped off at the quarry, a process known as *scalbling*. A piece of stone sawed or brought to a fairly good surface with a chisel is said to be *half-plain work*; those accurately done, that is, with the surfaces truly out of winding, are known as *plain work*.

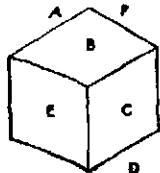


FIG. 271A.

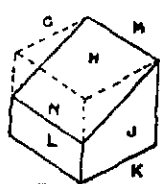


FIG. 271B.

Stones are often worked by rubbing them together, water and sand being added to assist the process, which is often done by machinery. It is known as *rubbed work*.

All chamfers, sills with sloping surfaces, grooves, etc., are called *sunk work*; and round work is known as *circular or circular sunk work* according as the surfaces

are convex or concave. A frequent point of difficulty to observant students is the following, of which an explanation is given:—

Why do not sunk and moulded work include labour to preparatory faces? * See figs. 271A and 271B.

In endeavouring to show why the labour to half-bed should not be deducted where sunk work is measured, it will be simpler first to find out why it should be deducted where plain face is measured, and then ascertain in what manner the conditions of sunk or moulded work vary from plain work. Imagine a block of stone to be converted into a cube, and consider the labours on it. Half-sawing would be necessary to all six faces A, B, C, D, E, F, but it would only be measured to faces having no other finish, namely, the back F, because it is covered by subsequent labours on all the other faces. Half-bed would be measured to top and bottom faces B and D, and half-joint to ends A and C. Plain face would be measured to front E. Thus all faces have been measured. Now if plain face is required to the whole or part of any of the other faces, it is clear that to make it equal in value to face E the preparatory labour (whether bed, joint, or sawing) previously measured must be deducted, because they are identical with and form part of the actual labour to plain face. Now imagine another cube prepared similarly to the previous one, namely, half-sawing to back M, half-bed to H and K, half-joint to G and J, and plain face to L, and assume that a splayed face N is formed from the back edge of H, terminating halfway down L. After the preparatory labours are worked to H, K, G, J, the surface H is destroyed and chipped away or sunk to the new plane of N and finished as a plain face. The finished area of N is measured as sunk plain face (or sunk face), and the remaining part of L (in this case one-half) as plain face. Half-bed to H and half-joint to parts of G and J are not deducted, because it is necessary that they should

* *Building World.*

be prepared before the sinking can be properly set out, and where the sinking is deeper than 3 in. a dimension of rough-sunk preparatory labour must be taken, and this further indicates that no deduction of bed or joint can be taken. The same applies to mouldings. Summing up these results, we find that half-beds, half-joints, and half-sawing (otherwise the preparatory labours) are deducted where plain face is measured over them because they form part of the plain face labour, but they are not deducted where plain or moulded work is sunk from them, because they have to be worked first, then destroyed, and a new surface formed in the required plane, so that they form no part of the subsequent labour.

Hardening of Soft Stone.

A process by means of which the softest sandstone or limestone can be made hard has been discovered by Mr A. Kubelka, of Bushowitz, in Moravia. The following are the methods he employs. First, the surface of the stone must be thoroughly cleaned, so as to expose the pores. Any oil or grease spots must be removed with benzine or with the alcohol flame. Missing pieces must be filled up with cement mortar, using a 1 : 7 solution of water-glass for tempering. After the stone is thoroughly dry, it is saturated with a solution of potash or soda waterglass. In case of rain during or immediately after this operation, the stone must be again cleaned, dried, and saturated with the solution. Then follows an impregnation with molten chloride of calcium. After this, rain will do no more harm, as on account of the reaction of the chloride of calcium upon the solution of waterglass the pores of the stone will be filled with insoluble, hard silicate of lime, while the soluble silicate of lime will be decomposed and washed out by rain. Another method of Kubelka's is to saturate the stone first with a solution of sulphate of alumina in water, and when dry with a solution of potash waterglass. Sometimes a repetition of this process will be necessary to thoroughly fill all the pores of the stone. It is enough if the stone is saturated to a depth of about $\frac{1}{4}$ in. Whitish brush marks may be cleaned either by rubbing with a piece of the same stone, or by refacing the stone, or by brushing with steel brushes. Should the solution not impregnate the stone quickly enough, it must be diluted with more water. The solution should be entirely absorbed by the stone in about one minute. Whatever is left over on the surface after this time should be wiped away with a rag, as the crystals formed by the evaporation of the water would make a rubbing and cleaning of the stone necessary. A separate brush should be used for each solution, so that the reaction should begin in the pores of the stone. For larger surfaces atomisers or sprinklers may be used successfully.

Through these processes any soft limestone or sandstone may be made more compact and hard. Their porosity and imperviousness will equal that of the hardest stone. After treatment they may be polished and cut. The great advantages of the soft stones—cheapness of quarrying and cutting—remain, and they can receive afterward all the prominent qualities of the harder stones.

Mortar and concrete may also be hardened and compacted by these processes, so that they may be used with greater success in works calling for water-tightness, as waterworks, tanks, street pavements, artificial stone, cement corks, etc. Monuments can also be protected by these processes from the fluence of the weather.

work out such calculations. For most walls we adopt a general formula for thickness.

t = thickness in ft.

L = length "

H = height "

c = a constant = $\frac{11}{20}$ for ordinary houses and $\frac{H}{12}$ for big warehouses.

$$t = \frac{Lc}{\sqrt{H^2 + L^2}} \quad (200)$$

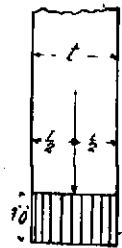


FIG. 275.

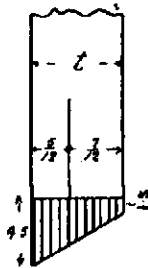


FIG. 276.

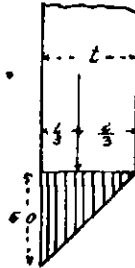


FIG. 277.

Another formula is as follows:—

$$T = \frac{HL}{CD} \quad (201)$$

t = thickness (ft.).

L = length (ft.) between party walls.

D = diagonal face of wall = $\sqrt{H^2 + L^2}$.

C = constant = 22 for dwelling-houses and 20 for warehouses.

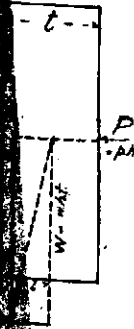


FIG. 278.



FIG. 279.

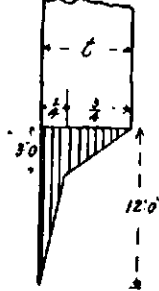


FIG. 280.

(walls would be one-third thicker.) These equations are useful for calculations, etc. which come before a municipal engineer for his approval. Builders often make them too thin.

Retaining-Walls.—We now come to what are called retaining-walls. They are of far more importance to the engineer than the foregoing.

Their use is resorted to very frequently for retaining cuttings and banks, abutments of bridges, sea walls, water walls, sides of tanks and reservoirs, reservoir dams, river walls, etc. They are all treated in more or less the same way for stability, under the heading of retaining-walls.

For retaining-walls we can get the required thickness in the same way as for ordinary walls, but of course we employ a different formula, because

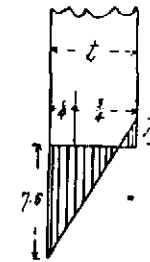


FIG. 281.

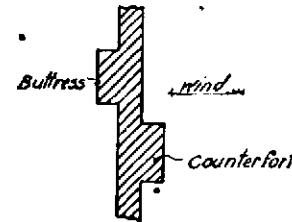


FIG. 282.

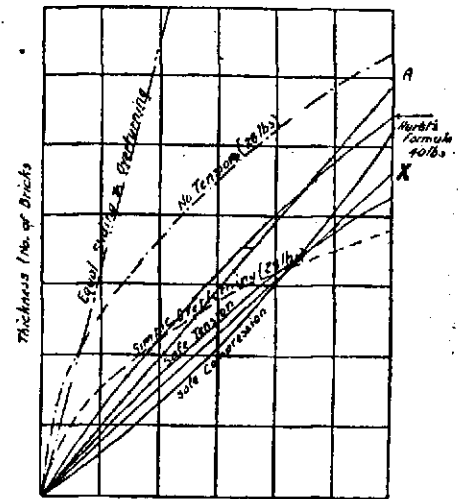


FIG. 283.

we have to take into account the weight of the earth backing and its angle of slope, which we will find in Chapter VI.

Now given

D = height of proposed wall in feet,

W = weight of a cubic foot of the wall (vide Table D), lbs.,

w = " " " earth at back (ditto), lbs.,

T = thickness of wall in feet also (mean thickness),

$$T = .7D \tan \frac{a}{2} \sqrt{\frac{w}{W}}$$

$\tan \frac{a}{2}$ signifies that we take the tangent of half the angle of repose. This

course, varies. It is about .414 for ordinary earth and .577 for sand. In practical calculations, however, we may be safe in taking it as .5. If, however, we are designing a wall for water, we must take it as 1.0. These remarks apply to walls which have the backing trimmed off quite level with the top of the wall and level. In many cases, however, it slopes upwards from the top—in fact, usually so. This, of course, will alter the results by increasing the load. They are then known as *surcharged revetments*.

To make use of Equation 202 (see fig. 284), set off $D = H$ and substitute

H_1 for H , as scaled off. When, however, the earth falls over the top of the wall to the front edge we account for it by measuring 1) from the point X , as shown by the same figure. In the same way as we advised in ordinary walls, long retaining-walls should have counterforts every 20 ft. or so. They are not of so much use in giving extra strength to the wall as they are in holding the earth backing together. For economical and other reasons, retaining-walls are sometimes built with a series of buttresses which are connected by arches, as in fig. 285. For the design of such walls we employ two formulæ. Using the lettering on the figure

$$\mathbf{L} = \mathbf{T} \times \frac{\sqrt{\mathbf{D}}}{\mathbf{B}} \quad (203)$$

$$\text{and} \quad \mathbf{B} = \mathbf{D} \times \frac{\mathbf{T}^2}{T^2} \quad (204)$$

while T = the thickness of the wall, would be found either by Equation 201 or 202 according to circumstances. Probably 202 would be used. A particularly good form of retaining-wall is shown in fig. 286 with a front batter. The thickness would, of course, be found by Equation 202, T being

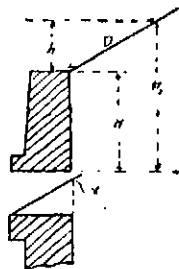


FIG. 284.

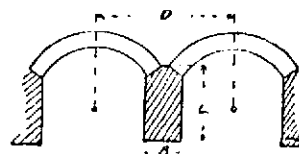


FIG. 285.

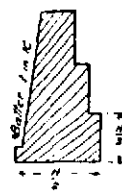


FIG. 286.

taken as the mean thickness at the middle; we would then draw it out with the required batter, say 1 in 10 (in any case it should never be more than 1 in 6), and make the back stepped as shown.

The use of the formulae which we have enumerated is resorted to when we want to design a retaining-wall for a particular purpose, inasmuch as we can draft out an *approximate* form of cross section, but they are only capable of general, not special application. The more usual way is, when a sketch design has been made, to find how the wall satisfies the principles of moments as applied to the stability of walls. It is this which we are now going to discuss.

In retaining walls the moment of stability is proportional to—

1. The heaviness of the material ($=w$ in equations).
2. The breadth and height ($=b$ and h " ").
3. The square of the thickness ($=t^2$ " ").
4. The inclination of the bed joints to the horizon.
5. The distance of the centre of resistance from the centre of gravity of the wall (horizontally).
6. The angle of slope of the material ($=\alpha$).

Now say in Equation 202 we found a certain value of T . This value applies to walls having a plumb face. When we have a batter matters change slightly. Thus, taking the straight wall as unity = 1.00, for a batter of 1

in 12 on the face it becomes 0.86, a batter of 1 in 8, 0.80, 1 in 6 = 0.74, and 1 in 5 = 0.72. But if the internal face is stepped, its value is 0.85 (fig. 286). Now figs. 287 and 288 show two forms of retaining-walls; one has the bed joints horizontal, the other has them distinctly inclined to the horizontal. In fig. 287, AW represents to scale the weight of the actual wall over a run of, say, 1 ft. *plus* the weight of earth which would be contained in the irregular space MOL. But in fig. 288, BOL is considered as equal to that in masonry, concrete, or other material, *less* such a quantity of earth. The pressure AP is always parallel to the surface of the bank, which is, of course, the natural slope of the ground, and is equal to the weight of the earth prism ODK, equal to

$$\frac{w_1 x^2}{9} \quad (205)$$

This quantity multiplied by the perpendicular distance from F to PH will give us the overthrowing moment of the wall = M. The reason that some

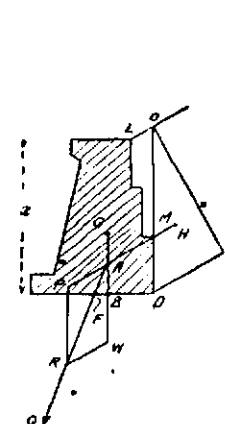


FIG. 287.

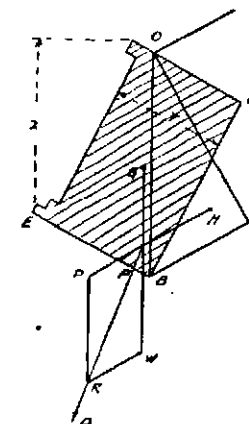


Fig. 288.

retaining-walls have a slanting base is to reduce the obliquity of pressure, and to allow the condition of frictional stability being fulfilled.

3. **Battering Walls.**—First set out a preliminary rectangular wall of suitable section, found by Equation 202, and find its centre of resistance = F (figs. 289 and 290). Now make $QN = 3$ times E/f^* and join EQ , then we have an irregular prism, and that is the part which may safely be removed, because the centre of gravity of the prism is directly over F .

Now to find the centre of gravity of such batter and curved walls, we proceed as follows. Find the centre of gravity of the complementary rectangular wall = G , then to find the distance $g - G$ we have two equations, viz.—

For battering walls $gG = \frac{1}{2}QN$ (206)

For curved walls $pG = \frac{2}{3}QN$ (207)

The curve EN may be either circular or parabolic in fig. 290.

* f is vertically below g in calculations.

Now the moment of stability of any wall is equal to

$$S = gwhlt^2 \quad (208)$$

($g = 375$ in English engineering practice).

Obviously, then, we must fulfil the condition that $S =$ moment of stability is equal to M , the overturning moment. The whole of our previous and subsequent reasoning depends on this condition, and to ensure that such is fulfilled is the engineer's business. When designing, both M and S should occur in lb. = ft.

Now when walls are built with large counterforts we have a slight saving of material in the end. Now say we have a wall with such counterforts placed x feet apart.

Let t = thickness of the wall between counterforts.
 " T = " " including counterforts.
 " y = width of the counterfort (see fig. 291).

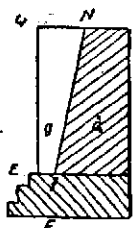


FIG. 289.

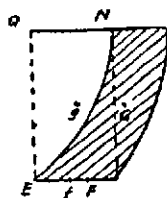


FIG. 290.

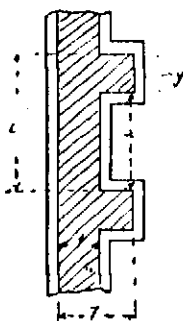


FIG. 291.

Then the moment of stability of the wall between the counterforts is equal to

$$S_1 = gwhxt^2 \quad (209)$$

and at the counterforts its value is

$$S_2 = gwhyT^2 \quad (210)$$

Now add these two moments together and divide their sum by the total length of wall = L . Then we have a formula giving the mean stability per unit length, and its value is

$$gwh \frac{xt^2 + yT^2}{x + y} \quad (211)$$

By transposing we have

$$t_1 = \sqrt{\left(\frac{xt^2 + yT^2}{x + y} \right)} \quad (212)$$

and this may be compared with our previous reasoning to see what saving of material we have. Professor Rankine terms this the *equivalent uniform wall*.

The ratio of masonry in a counterforted wall is to the masonry in a plane wall

$$\text{as } (xt + yT) \text{ is to } (x + y)t_1 \quad (213)$$

The saving is *very small*. The foundations of retaining-walls are usually stepped out in front. The object of so doing is to bring the moment resistance towards the middle of such a base. Ashlar or block in course (vide previous chapter), when backed with rubble, are specially suited to retaining-walls. Long headers or bond stones are, however, essential. Regarding the drainage of the back of the walls, we may construct a drain as fig. 317. Weep holes should be provided too, one for every 4 square yards wall surface. They should be 2-3 in. wide, and equal in depth to the counterforts they lie in. Some engineers provide a gravel backing; it must be very carefully rammed, as the work proceeds, in layers. If, however, the backing is stiff clay, there should always be a drain as in fig. 317 provided. It is called a "blind drain" (vide Land Drainage), and it would be constructed of concrete or gravel. If, however, the ground is very wet, it will be advantageous and prudent to design it as a *water wall* (Equation 202). In soft ground also process called *under-setting* is resorted to (see fig. 69), or we may have

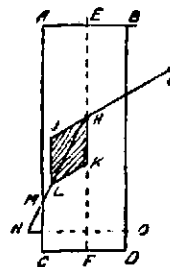


FIG. 292.

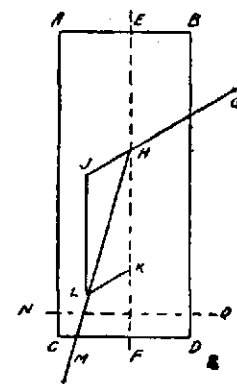


FIG. 293.

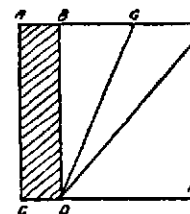


FIG. 294.

design the wall as in fig. 285. The walls of No. 9 Dock of the Manchester Ship Canal were so designed. Now figs. 292 and 293 represent the equilibrium of retaining-walls. A wall may fail by—

- (1) Shearing.
- (2) Toppling over.
- (3) Crushing on the outer joints, and consequent separation of the inner ones.
- (4) Sliding on the surface.
- (5) Sliding on the bed joints.

GJ = line of pressure.

HK to scale = pressure due to the wall.

HJ = thrust due to earth load. If now the angle MHF = the limiting angle of resistance, which is the angle at which the earth starts to slide up itself, the coefficient of friction being the tangent of this angle, the wall will either slide upon the joints or on the ground if MH does not fall in the base of the wall at all (fig. 292). But if, as in fig. 293, it *does* fall in the base of the wall, it is safe against such sliding, but not altogether safe until the line falls in the *middle 3rd* of the base. Now in fig. 294 ED = natural slope of ground, and EDF = the limiting angle of resistance, AECD = true wall. Now BG is made equal to $\frac{1}{2}$ BE, and the pressure of the earth up

the wall, as we pointed out, is equal to $\frac{1}{2}BG^2 \times$ the weight of 1 cub. ft. of earth, say 110 lbs.; this gives us what is known as the *active pressure*. It is now obvious to the reader that the earth which is behind the angle of repose does not affect the stability of the wall in the least, but the wedge in

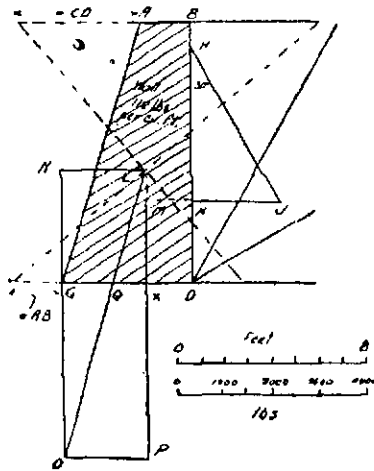


FIG. 295.

between is that which takes effect only. By bisecting the wedge we have what is termed the *line of rupture*. In the fig. 294 the angle of repose is taken as 30° , therefore GD bisecting it gives us the line of rupture, and BGD is the wedge of earth which we have to calculate upon. Calculate the weight of the earth and calculate the weight of the wall, assuming both to be 1 ft. wide (vide fig. 295). Make KD $\frac{1}{3}$ the way up from the base, and to scale make KII = weight of the wall. Now make the angle KHJ = 30° (or the angle of repose), and produce it to meet the perpendicular from K. Now JK to scale = force acting on the wall in lbs. Now find the centre of gravity of the wall by any known method. A simple way is by means of the dotted lines, producing the top each side a distance equal to the base, and producing the base likewise a distance = to the top as shown on the drawing. The centre of gravity will then be at the intersection of the two diagonals at L. Drop a perpendicular LP making MP = weight of wall, in this case = 3920 lbs., complete the parallelogram, MO is then the all-important line, the line of resistance of the wall, which must pass through the *middle third* of the base if the wall is to be safe.

The above graphic construction is of the utmost value and importance to the engineer. It is simple, but must be very accurately drawn to a large scale. If this be done, no failure of the wall from *natural* causes is likely to occur. Its neglect in design may mean complete failure; and if not extravagant use of material.

When the ground has a sloping surface such as occurs in what we termed *surcharged revetments*, the above graphic construction holds good, because it simply means calculating a larger wedge of earth. For the sake of comparison we will give another instance of graphic statics as applied to earth retaining walls. The wall, which is not rectangular, is shown in fig. 300.

AB represents the inner side of the wall. AR = slope of ground.

On AR take any point M and draw MO perpendicular to AR making MO = AB. Now draw OS, making α = angle of repose, bisect the angle GMR by NM. Then NM is the direction of the *principal stress*. From B draw BD at right angles to AB and BC parallel to NM, and from D draw DC = DB and cut off DF = MR. Join FB, and this equals the *intensity*

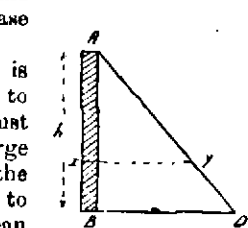


FIG. 296.

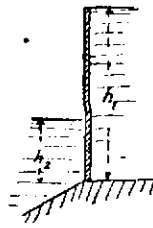


FIG. 297.

of the resulting stress. Now make BE = BF, join EA and the area of the $\triangle ABE$ = area of pressure against the wall AB, and this multiplied by a unit volume of earth will give the *total pressure*. It acts at the point H. Find $\frac{1}{3}$ of gravity of the wall, and drop GP perpendicular, draw KH parallel to BF and produce to L, making KL to a suitable scale = thrust of the area ABE, and make KP to the same scale = weight of the wall, complete the parallelogram and join KJ. KJ is then the direction of the resultant forces on the wall; it must pass through the middle third; if too far out, the wall would *opple* over; if too far in, it would be *thrust* out at the toe.

Now the previous reasoning has been mainly confined to treating walls as subject to earth pressure only. Another very important branch of the subject of walls are those which have to withstand the pressure of water. Under this heading come dock walls, reservoir dams, walls of tanks for sewage and water, and various other hydraulic works, such as turbine pits, weirs, etc.; refer to fig. 296. Here we suppose the wall AB to be subject to the pressure of water on the right-hand side. AB = depth of liquid = h which is = 62.5 lbs. per cubic foot (see also Hydraulics); then by making $3D = AB$,

$$P = \frac{3D \times AB \times w}{2} \quad (214)$$

will give the *total pressure* on a piece of the wall 1 ft. wide. Again, at any point x the pressure is equal to

$$\frac{Ax \times xy \times w}{2} \quad (214a)$$

The moment tending to overthrow the wall is equal to

$$\frac{wh^3}{6} \quad (215)$$

Now this moment will act at a point $\frac{1}{3}$ the way up the wall from the bottom of the liquid; this point is called the *centre of pressure*. Take, however, the case of a wall or sluice gate in which we have water on each side at different levels; see fig. 297. Here we have slightly different calculations to contend with. Now the total pressure on the gate is found by the equation

$$h_1^2 - h_2^2 \left(\frac{bw}{2} \right) \quad (216)$$

b being the breadth of the gate. All measurements are in feet, but this pressure will no longer act $\frac{1}{3}$ the way up the gate (or wall), but it will act at a point

$$\frac{\frac{1}{3}(h_1^3 - h_2^3)}{h_1^2 - h_2^2} \quad (217)$$

up the wall. The latter equation is particularly useful to the engineer to enable him to find the point of the hinge of a swinging gate (called a *reflux gate*) used in land drainage and town sewerage schemes, especially on tidal outfalls, which is required to open when the water reaches a certain height on the inside or outside. The pressure would, of course, act at the centre of such a gate; the weight of the gate, of course, is taken into account. It constitutes an opposing moment to that found in Equation 216.

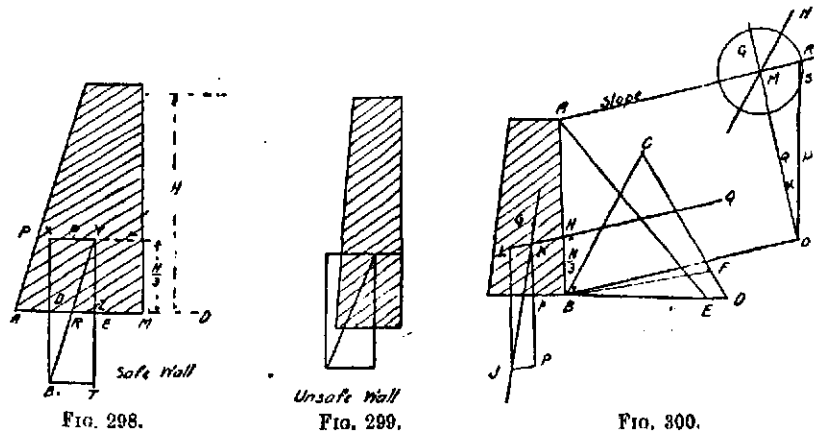
Again, Formula 215 gave us the overthrowing moment of the wall. This,

of course, also must have an equal and opposite resisting moment. Given W = weight of wall in lbs. per foot-cube; then the resisting moment is equal to

$$\frac{Whb}{2} \quad (218)$$

Again, we can apply graphic statics conveniently to water walls. Figs. 298 and 299 show two cases of a safe wall and unsafe wall. In the same way as previously described LM must be set out = $\frac{1}{3}H$. YT = weight of the wall, must pass through the centre of gravity of the wall. P = pressure of the water (Equation 214). Then when YB passes through the middle third of the base as in fig. 298, the wall will be safe, but if it occurs as in fig. 299 it will not be safe.

We will now turn to the consideration of reservoir walls, which are straight or plan-curved walls, and require more elaborate treatment. For



all such walls we have a fundamental equation which will become very familiar to every engineer in the course of his studies. It stands as

$$\frac{W}{A} \pm \frac{M}{Z} \quad (219)$$

By its use we can find the maximum tension or compression set up in the joints of the wall in question. These must, of course, be within safe limits. Now

W = the weight of the wall in lbs. (or cwt., tons, etc.) per foot-run.

A = the area of the base of the wall.

$M = Wx$ by the distance from the line passing through the centre of gravity of the wall to the inner edge ($=x$, fig. 301).

$Z = \frac{1}{3}bd^2$.*

Now if we work out the two sides of this equation, and add the two values so found (the sign $\pm$ means that we can either add or subtract them), we will get the amount of compression which will occur on the outer face of the wall, whereas if we subtract them we will have the tension on the inner joints likewise. Of course it is the business of the engineer to see that the wall is

* = modulus of section of a rectangle.

designed to give safe limits in tension and compression, and it is for this reason that we have such an equation, so as to be able to check over our previous calculations. Naturally the tension should be very small, especially in brick-work and masonry, where the adhesion of the joints is the sole resistance to tension. In a homogeneous mass such as concrete we may allow a slightly higher value, about 45 lbs. per sq. in. For the safe compression see Table D at the end of this chapter. Let us take some practical examples of the foregoing reasoning.

✓ For instance, assume we have to design a wall to support the side of a road, a very likely case. It is to be built in masonry set in cement. Say

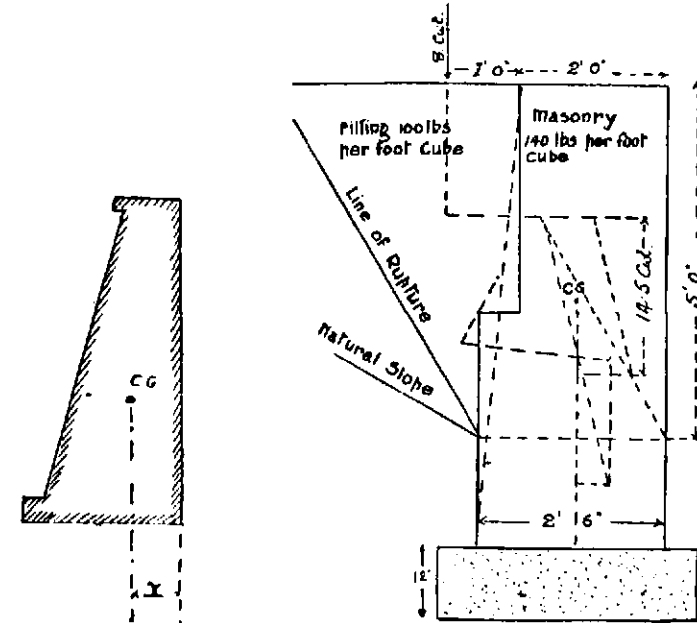


FIG. 301.

FIG. 302.

its outer face must be vertical (for some particular reason), and the greatest load we may expect on the road will be a 10-ton roller. Strictly speaking, this would not come under the heading of reservoir walls, but being a very good example of what might any day confront the reader, we give it here. The load acts at a minimum distance of 3 ft. from the face of the wall; this would give a distribution of $\frac{8}{20} = 0.4$ ton, or 8 cwt. per foot-run. The section

of wall as shown in fig. 302 may then be assumed, and the pressure from the filling combined with the weight of the wall. This resultant is then combined with the thrust from the roller, giving the final resultant which cuts the base at 1 ft. 4 in. from the centre. The stresses produced will then be

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{14.4}{2.5} \pm \frac{14.4 \times 1.3}{\frac{1}{3}(1 \times 2.5^2)} = 5.76 \pm 17.9$$

weighs $2 \times 62.5 = 125.0$ lbs. Required the shearing force acting in a vertical plane which is 10 ft. from the outer toe. We proceed as follows:—

Weight of wall per foot-run,

$$\frac{100 \times 70 \times 125}{2} = 437500 \text{ lbs.}$$

Then the intensity of pressure at the outer toe, in order to give zero at the inner toe, will be

$$\frac{437500}{70} \times 2 = 12500 \text{ lbs. per sq. ft.,}$$

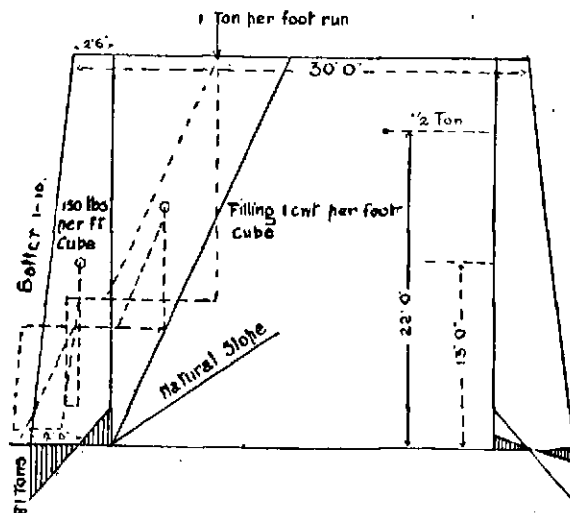


FIG. 306.

and at 10 ft. from it

$$12500 \times \frac{60}{70} = 10714.3 \text{ lbs. per sq. ft.}$$

If the conditions are correctly stated, the resultant of the forces will cut the base at the edge of the middle third. The thrust from the water will be

$$\frac{1}{2}wh^2 = \frac{1}{2} \times 62.5 \times 100 \times 100 = 312500 \text{ lbs.}$$

acting at one-third the height, and the weight of the wall per foot-run has been found to be 437500 lbs. Then

$$Px = Wy, \text{ or } y = x \times \frac{P}{W} = 33\frac{1}{3} \times \frac{312500}{437500} = 23.81 \text{ ft. ;}$$

but $\frac{70}{3} = 23.33$ ft., which is not quite exact for the middle third, but sufficiently near. Now the vertical shear at a point 10 ft. from the outer

* x = moment of P and y = moment of W .

toe will be the whole of the reaction between the toe and that point, minus the load over the same distance

$$\begin{aligned} &= \frac{12500 + 10714.3}{2} \times 10 - 10 \times \frac{100}{70} \times 125 \times \frac{1}{2} \\ &= 116071.5 - 8928.6 = 107142.9 \text{ lbs.} \end{aligned}$$

As the height of the section at this point will be 14.3 ft., the stress per sq. ft. will be

$$\frac{107142.9}{14.3} = 7493 \text{ lbs.} = 3.35 \text{ tons per sq. ft.}$$

The conditions will be as shown in fig. 307.

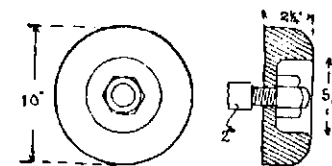


FIG. 308.

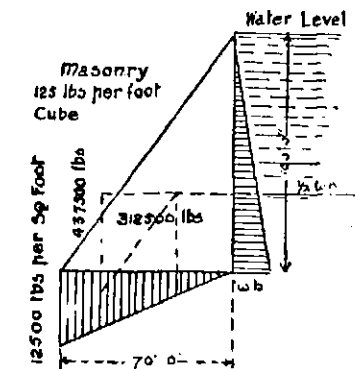


FIG. 307.

Another interesting and instructive example is that of a reservoir being reduced in area by a division wall. It is to be 105 ft. long, and laid across the entrance to one arm in continuation of one of the main walls. The arm is afterwards filled in, and the roadway made over it. The dimensions of the existing wall are to be repeated at A in the figure, and the dimensions of the inner wall B are to be determined. If the wall A is sufficiently heavy by itself to resist the thrust from the earth under the street, only a wall of very small section would be required at B. It would hardly be subject to calculation. First draw out the full section as shown, and find the thrust at the back of the rubble wall. This, combined with the weight of the wall, gives us the first resultant. Allowing for the weight of a steam roller, being about the greatest load likely to come upon the road, we combine it with the previous resultant and get a fresh one. Then using our familiar formula (No. 219),

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{118}{8} \pm \frac{118 \times 0.8}{\frac{1}{8}(1 \times 8)^2} = 14.75 \pm 8.85 \text{ cwt.} = 23.60 \text{ cwt.}$$

= 1.13 tons per square foot compression on the water side of the wall and 5.90 cwt. or 90.2 tons per square foot compression on the other side. This latter assumption is, however, based on the fact that the reservoir is empty.

= 23.66 cwt. or 1.183 tons per sq. ft. compression, and 12.14 cwt. or 0.607 ton per sq. ft. tension, which are within safe limits. A very considerable saving could be obtained by battering the face of the wall, if that were permitted.

Now let us take the case of a small reservoir built on a hillside, such as would supply a large mansion with water. We will assume it to be built in brickwork set in cement mortar, and rendered inside. The slope of the ground might be 1 in 4, and the size of the reservoir 65 ft. long $\times$ 10 ft. wide (mean width) and 10 ft. deep.

Fig. 303 shows the reservoir walls and the necessary diagrams drawn to scale. Having drawn such a diagram, the thrust on the surcharged wall due to earth pressure will be found as follows. Set up a line in the direction AB, at the same angle from the vertical as the natural slope of the earth. Produce the slope of surcharge to meet it, and bisect the distance between

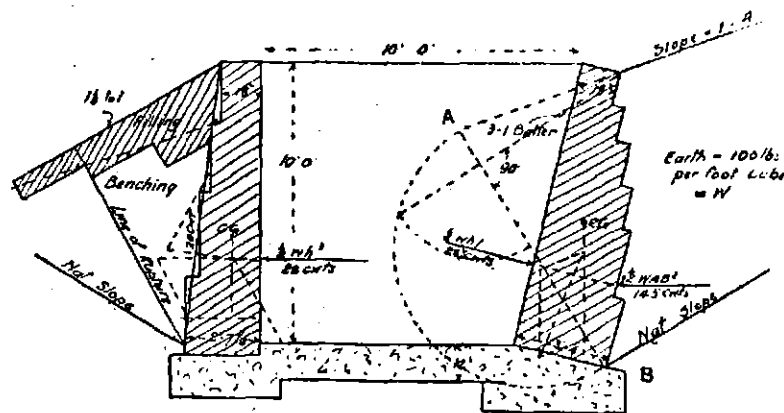


FIG. 303.

the intersection and the base of the wall, and describe a semicircle on it. From the foot of the surcharge at the top of the wall drop a perpendicular on to the line AB, and produce it to meet the semicircle. Then from the end of the semicircle with radius to this last point, describe an arc cutting AB. Then the horizontal thrust at one-third of the height of the wall will be

$$\frac{1}{2}w(AB)^2 = \frac{\frac{1}{2} \times 100}{112} (5.7)^2 = 14.5 \text{ owt.}$$

Combining this with the weight of the wall, 25.6 cwt., the resultant is given, which cuts the base of wall at 0.7 ft. from the centre of the base.

When empty the maximum stresses will be

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{29}{3.375} \pm \frac{29 \times 0.7}{\frac{1}{8} \times 1 \times 3.375^2} = 8.6 \pm 10.7$$

= 19.3 cwt. or 0.96 ton per square foot compression, and 2.1 cwt. or 0.1 ton per square foot tension.

When full the water will press on the face of this wall perpendicularly at one-third the height with a force of

$$\frac{1}{2}whl = \frac{1}{2} \times \frac{62.5}{112} \times 10 \times 10.3 = 28.7 \text{ cwt.}$$

This would be combined with the previous resultant, giving a new resultant cutting beyond the inner edge of the base, and giving the stresses

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{32}{3.375} \pm \frac{32 \times 2.4}{\frac{1}{8} \times 1 \times 3.375^2} = 9.5 \pm 40.4$$

= 49.9 cwt. or 2.5 tons per square foot compression, and 30.9 cwt. or 1.54 tons per square foot tension.

On the lower side of the reservoir the thrust of the earth is taken in the usual manner for retaining-walls, giving the maximum stresses as

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{21}{2.625} \pm \frac{21 \times 1.8}{1 \times 2.625^2} = 8 \pm 33$$

= 41 cwt. or 2.05 tons per square foot compression, and 25 cwt. or 1.25 tons per square foot tension. When full, the water pressure on the other side reverses the stresses and gives

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{21}{2.625} \pm \frac{21 \times 2.75}{1 \times 1 \times 2.625^2} = 8 \pm 50$$

= 58 cwt. or 2.9 tons per square foot compression, and 42 cwt. or 2.1 tons. per square foot tension. All these are safe stresses; therefore the walls may be considered satisfactory.

Assume we require to design a wharf wall of the form shown in fig. 304. In fig. 305 we give an enlarged view of the walls, showing the various diagrams and other calculations. When calculating the left-hand side we have to take into consideration a line of rails over which a train passes. We allow 1 ton per foot-run for this, as shown in the fig. Then, using Equa-

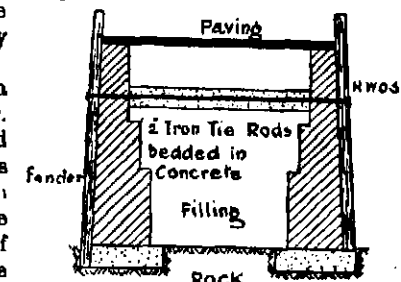


FIG. 304.

tion 219, we have our plotted values of 8.1 tons compression and 5.3 tons tension. These are both per square foot. Obviously our tension is far too much, and hence we put in tie rods, as shown in fig. 304, placed 20 ft. apart, and 2 in. in diameter; if they were $1\frac{1}{2}$ in. in diameter and 10 ft. apart they might be used to more advantage, perhaps. Fender piles, of course, are provided; and where the tie bolts happen to come through them, or where bolts are provided to fix them to the wall, we would need to resort to the use of special washers made of cast iron. A good form is shown in fig. 306. The piles are 10 ft. centre to centre.

Besides the stresses we have just been considering, we also have to calculate upon a shear stress on such masonry, dams, etc. Take a triangular masonry dam as shown in fig. 307. It is assumed to be 100 ft. high, having a vertical face inside. The water-level is at the extreme top of the wall. It varies in width uniformly, and has a base width of 70 ft. The specific gravity of the masonry is 2. ∴ a cubic foot of the masonry

then, by the theory of arches, when w = weight of 1 cub. ft. of water = 62.5 lbs., and if R and h are as in fig. 311, the horizontal thrust at every point on that particular curve = Rhw (223)

If now we are to limit our maximum stress in the masonry to 8 tons per sq. ft., it will be obvious to the reader that the thickness of the dam at the point in question

$$= R - r = t \text{ ft.}$$

and

$$t = R \left(1 - \sqrt{1 - \frac{2hw}{S}} \right) \quad (224)$$

(Waterworks, Tedsbury and Brightmore.)

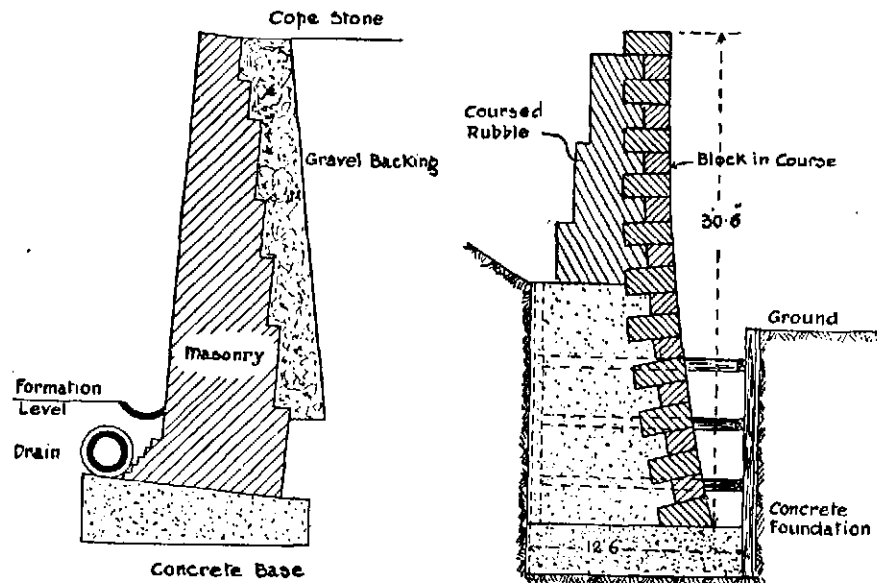


FIG. 312.

FIG. 313.

Say, now, $R = 100$ and $h = 100$, $w = 62.5$ and $S = 10$ tons per sq. ft.,

$$\begin{aligned} t &= 100 \left(1 - \sqrt{1 - \frac{2 \times 100 \times 62.5}{10 \times 2240}} \right) \\ &= 100(1 - \sqrt{.444}) = 100(1 - .657) \\ &= 100 \times .343 = 34.3 \text{ ft.} = t. \end{aligned}$$

We shall return to the subject of reservoir walls under Waterworks.

To conclude the chapter, we will give some actual practical examples of wall design.

Fig. 312 shows a typical form of retaining-wall for a railway cutting with a gravel backing.

Fig. 313, a large dock wall, showing the preliminary timbering.

Fig. 314, a quay wall for a minimum depth of water of 21 ft.

Fig. 315, a canal wall.

Fig. 316, a quay wall, illustrating the use of bags of cement. All these are actual practical examples.

Reinforced concrete is now becoming a recognised means of economically

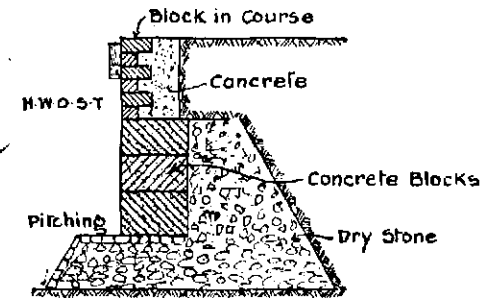


FIG. 314.

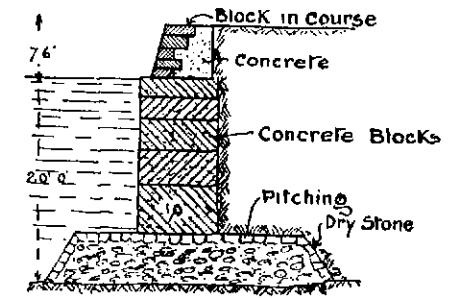


FIG. 315.

constructing retaining-walls and dams, especially those having neither counterforts nor buttresses. The first thing to do is, of course, to determine the pressure at the back of the wall, in the manner already set forth.

That is to say, assuming the wall 21 ft. high from foundation level, and earth at 100 lbs. per ft. cube, the thrust at the base = $\frac{1}{2} \times 21 \times 100 = 700$ lbs., which of course diminishes directly as the height, the total thrust being equal to $\frac{21}{2} \times 700 = 7350$ lbs. over 1 ft. of

width, because we consider the pressure to diminish uniformly to nil at the top from a maximum of 700 lbs. Consequently, by constructing a triangle base = 700 lbs. and height 21 feet to a suitable scale, the thrust at any point may be read off, the centre of pressure being $\frac{1}{3}$ the way up the wall as usual, viz. 7 feet. $\therefore$ the wall when considered as a cantilever fixed at the base, we have $7350 \text{ lbs.} \times 7 \text{ ft.} = 51,450$ lb.-ft.

In the same way the bending moment can be found at any other point. We now work out a series of values for the BM and for the thrusts, and plot a pressure diagram for our wall, which is best done by setting off the line of loads in the form of a polar diagram, already described, considering the wall as divided into ft. divisions, the centre ordinate of each division being considered. We then can construct a diagram for loads and bending moments by drawing lines parallel to the polar vectors between the ordinates which pass through the centre of gravity of the divisions on the pressure

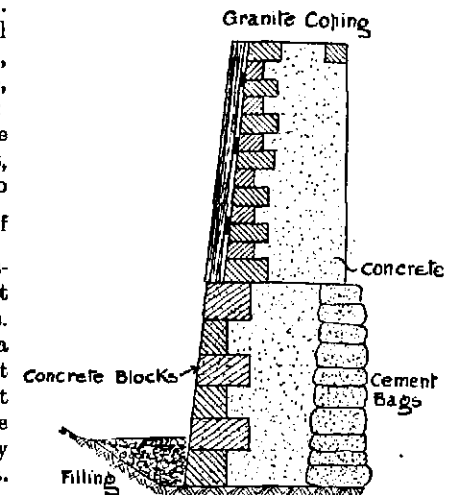


FIG. 316.

diagram, the bending moment in by the distance of the pole from the line of loads in feet.

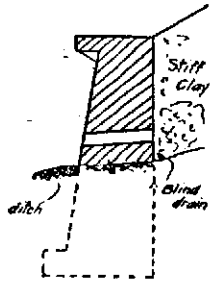


FIG. 317.

Let us now consider the actual design of the wall. Let b = breadth of wall in question, viz. 1 ft.; d = distance from compression surface to centre of reinforcement; c = maximum compression allowed in concrete, say 600 lbs. per sq. in.; t = maximum tension allowed in steel, say 17,000 lbs. per sq. in., while a = area of steel in sq. in. Assuming the wall 2 ft. thick at the base and 2 in. to cover the concrete, $d = 24 - 2 = 22$ in.

We now proceed to find out what would be an economical thickness for the base of the wall, assuming that the face has a straight batter of $\frac{3}{4}$ in. to the foot, leaving it $8\frac{1}{2}$ in. thick on top. First we proceed to find the value of the coefficient n = distance from

compression surface to neutral axis,

$$= \frac{Ecd}{t + Ec} = \frac{15 \times 600}{17000 + (15 \times 600)} \\ = .364; \therefore n = .364d$$

or if we assume the wall 2 ft. thick at the base, we can also find n on this assumption, so—

$$ta\left(d - \frac{n}{3}\right) = \text{the moment of tension.}$$

$$\therefore n = \sqrt{\frac{30ad}{b} + \left(\frac{15a}{b}\right)^2} - \frac{15a}{b} \\ = \frac{30 \times 2 \times 22}{12} + \left(\frac{15 \times 2}{12}\right)^2 - \frac{15 \times 2}{12}$$

(assuming 2 in. of covering to reinforcement bars when considering d)

$$= \sqrt{110 + 6.25} - 2.5 = \sqrt{116.25} - 2.5 \\ = 10.77 - 2.5 = 8.27;$$

and as $24 \times .364 = 8.73$, we find the two values differ by about $\frac{1}{2}$ in., due to d being assumed. We shall use the first value.

Now the moment of compression is equal to

$$\frac{bcn\left(d - \frac{n}{3}\right)}{2} = \frac{2BM}{bc} \\ = n\left(d - \frac{n}{3}\right) \therefore \frac{12 \times 51450 \times 2}{12 \times 600 \times .364} \\ = n\left(d - \frac{.364d}{3}\right) = d^2 - .1213d^2 \\ = 1 - .1213d^2 \therefore d^2 = \frac{471}{1 - .1213} \\ = \sqrt{535.3} = 23''$$

* For explanation of the notation used here the reader is referred to the chapter on Reinforced Concrete.

as the thickness of the base of wall, which differs by 1" from our assumed calculation to find n .

The next thing is how much steel shall we have.

Now

$$BM = ta\left(d - \frac{n}{3}\right)$$

$$A = \frac{12 \times 51450}{17000 \times \left(23 - \frac{.364 \times 23}{3}\right)} \\ = \frac{12 \times 51450}{20.21} = \frac{15435}{8639.25} = 1.8$$

sq. in., which would be well provided by 4. $\frac{3}{4}$ in. round bars per ft. of wall

$$= 4 \times .443 = 1.772 \text{ sq. in.}$$

The wall is shown in section in fig. 317A. We have now to consider the base or foundation slab, which provides an anchorage to the wall to prevent overturning.

It must be treated in the same way as the base of any ordinary wall for overturning, that is to say, the resultant of the three parallel forces must fall within the middle third of the base if a uniform pressure over the whole is aimed at, which of course is essential to stability. It is usual to place the wall towards the earth side of the slab, because if at any other part there may be undue excavation and timbering required at the point A. It may, however, be in the centre, or near either edge; in any case, however, the resultant must not fall—

(a) outside the slab (or else the structure will overturn);

(b) outside the middle third (or else there will be no pressure on the soil at A). We shall take 10 ft. as the assumed width of base.

Find the centres of gravity of the slab and the wall, and the tail end of slab, and through them draw lines 1—2 and 3. Allowing the concrete 145 lbs. per foot cube, find the weight of wall and slab, and on 1 set off the weight of the wall, and on 2 set off the weight of the slab, and join the diagonals as shown.

Where they intersect, drop a perpendicular, 4.

Now calculate the weight of the tail end of the wall and set it off on 4 to the same scale as the others on 1 and 2, while on 3 set off likewise the combined weights of 1 and 2, and join up the diagonals, whence we have the point R, through which we draw a line vertically to meet the thrust line at R₁. The position of the resultant is found in the manner already described, and as shown in the diagram. It must fall in the middle third.

The next calculation is, find the max. pressure at the edge of the slab, which is equal to

$$\frac{2x}{3y} = P,$$

which must be within safe bearing limits of the soil, or else the dimension L must be increased. Provided this is so, we require to find the max. stress to be resisted by the reinforcements

$$P \times \frac{z}{2} \times L - y,$$

where y , z , and L are in inches. Whence the value of n is found in the same way as for the wall.

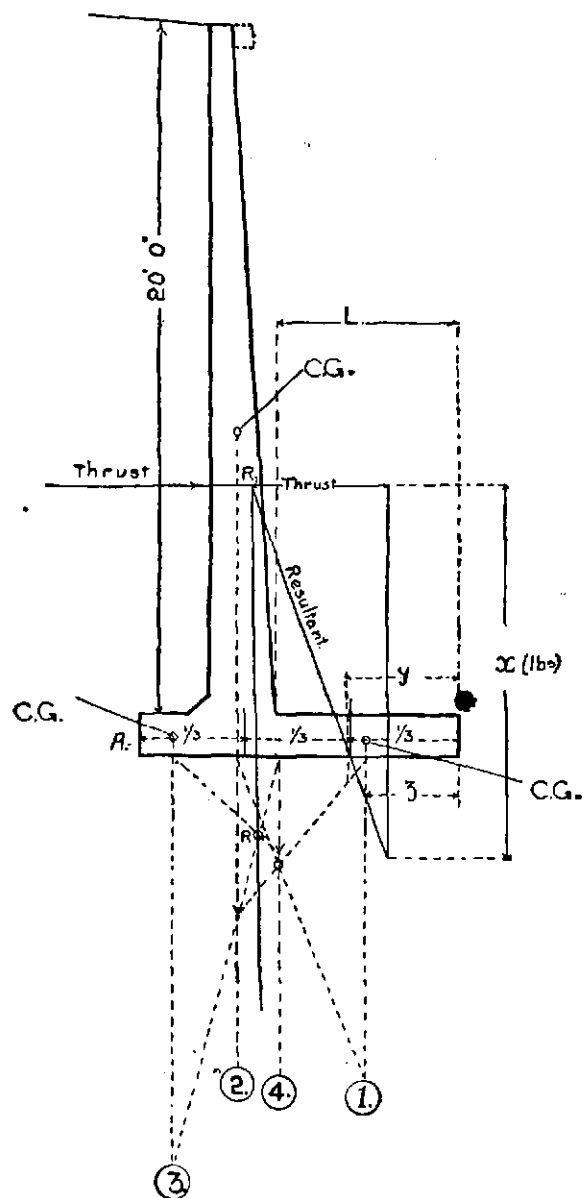


FIG. 317A

TABLE A.—STABILITY OF WALLS.

Wind pressure, lbs. per sq. ft. on a plane surface.

Height in Feet.	Length in Feet.						
	5.	10.	20.	50.	100.	200.	500.
150	54·6	50·3	46·3	41·4	38·1	35·1	31·4
100	48·0	44·2	40·7	36·4	33·5	30·8	27·6
50	38·4	35·4	32·5	29·1	26·8	24·7	22·1
20	28·7	26·4	24·3	21·7	20·0	18·4	16·5
10	23·0	21·1	19·5	17·4	16·0	14·8	13·2
5	18·4	17·0	15·6	13·9	12·8	11·8	10·6

25 per cent. to be added in very exposed positions.

TABLE B.—SAFE THICKNESS OF BOUNDARY-WALLS.

Height in Feet.	Wind Pressure. Lbs. per sq. ft.	Number of Bricks thick
5	14.04	1 brick.
6	14.56	1 "
7 5	15.22	1½ "
10	16.18	2 "
12	16.87	2 "
16	17.09	2½ "
18	18.18	3 "
20	18.52	3 "
25	19.82	4 "
30	20.09	5½ "

Figures plotted in fig. 283.

TABLE C.—WIND FORCE.

Description.	Velocity, M.P.H.	Corresponding Pressure per sq. ft. (lbs.).
Light breeze . . .	5	$\frac{1}{8}$
Good breeze . . .	10	$\frac{1}{4}$
Strong breeze . . .	15	$\frac{1}{2}$
High wind . . .	20	2
Half gale . . .	30	$4\frac{1}{2}$
Whole gale . . .	50	$12\frac{1}{2}$
Storm . . .	60	18
Hurricane . . .	80	32

TABLE D.—STRENGTH AND WEIGHT OF MATERIALS.

Material.	Safe dead Load. Tons per sq. ft.	Factor of Safety.	Weight, lbs. per ft. cube.
Granite masonry	15 tons	20	160
Portland and hard limestone	15 "	15	140
Sandstone	12 "	10	130
Blue brick (in cement)	9 "	8	120
Stock brick	6 "	5	115
" Lias mortar	5 "	4	112
" Grey "	3 "	4	112
Portland cement concrete	5 "	8	130
Lias lime concrete	3 "	8	120
Compact earth	2 "	...	112
Made ground	1 "	...	110

CHAPTER XVII.

CONSTRUCTIONAL CARPENTRY.

CARPENTRY is the art of constructing works in timber. The art, of course, more or less belongs to the carpenter and builder, but the engineer's services are required when works of larger magnitude than dwelling-houses are concerned, and it is in connection with such works that the general principles of carpentry will now be discussed.

The engineer should understand the principles of design, the most common joints and fastenings, and the most economical and safe use of the various

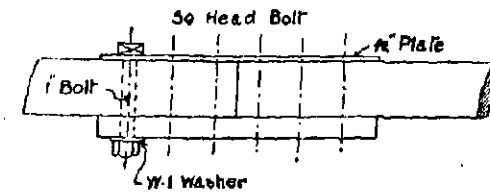


FIG. 318.

materials employed. The heavy carpentry work to be described specially runs a civil engineer in municipal practice. It includes—

Large timber roofs.

Arch centerings.

Shoring and Strutting.

Timber Bridges.

Piles and Pile driving (before described fully).

Timbering of Trenches.

The first thing to do is to investigate the properties of good timber. These are set forth in Chapter XIII. on Materials of Construction, and will not be repeated here.

The first thing to consider are theoretical points of design in general, and particularly the joints and fastenings employed on large timber structures.

First of all, we have the common fish joint (figs. 318 and 319). The two beams are joined abut against each other, and on each side are plates of wood (usually wrought iron), with bolts passing through them. This joint is economical both of labour and material, and is much used in temporary structures. In fact, it may be said that in Britain any works of carpentry in which an engineer will be called upon to design will probably be of a

temporary nature, such as a bridge to take traffic during the rebuilding of an existing one, temporary bridges to take light railways used on large works such as reservoirs, and gantries for cranes on big masonry works like reservoir dams.

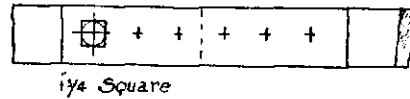


FIG. 319.

Timber is more or less liable to shrink, and such shrinkage will cause much stress on the bolts. To avoid this, hard-wood keys are often inserted as well as the bolts, as shown in fig. 320. Folding wedges are also used for the same purpose. When designing, always make the sectional area of the fish pieces equal to that of the tie, and provide bolts having a combined total sectional area = $\frac{1}{2}$ of the sectional area of the timber after the bolthole have been cut.

The second variety of joint is what is called a scarfed joint. It is both a neat and effective joint, and the usual proportions in terms of the depth are shown in fig. 321. bc is what we call the detrusion area = $7ab$, while bc

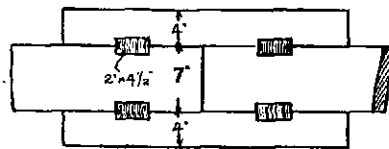


FIG. 320.

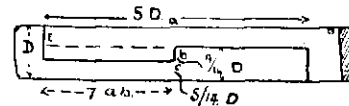


FIG. 321.

is the compressile area = $\frac{1}{4} D$, while ab and cd are the tensile areas = $\frac{1}{4} D$. The failure of these joints shows under what particular stress the joint has failed.

Bolts and fish plates may also be used. They may also be assisted by oak wedges. The duty of such wedges is to provide for shear stress. In a long span we have to build up the beams. Fig. 322 shows the method of procedure. Two beams are placed over each other; they are fastened by iron straps, and have hard-wood keys as shown. A more elaborate method is shown by fig. 323. It is very costly, however. Another joint not so common in engineering works is that called the tusk tenon. It would be employed for such purposes as fitting the cross joists to a timber bridge. It is shown dimensioned in terms of the depth in fig. 324.

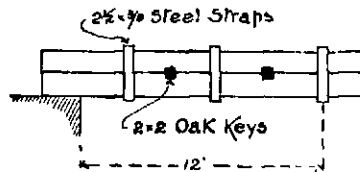


FIG. 322.

Now it is essential that the joints in all beams should be parallel to the load upon them (viz. usually vertical). The tenon which we have just described fits into the other member by means of a slot called a *mortise*. These mortises are best made by hand, but are now largely cut by machinery, and are usually known by the ends being as in fig. 325 instead of exactly square. Another joint sometimes used on large beams is the splayed scarf. It is tightened up by means of folding wedges, as shown by fig. 326.

As regards strengthening wood beams, what is termed a flitch beam was described fully, with calculations, under Beams. There were also discussed in the same chapter some calculations regarding trussed wood beams, and

we treated them graphically in "Graphic Statics." The object of trussing a beam is to increase the bearing power of any particular section by providing iron or steel stay rods to take up part of the tension while the timber itself takes up the compression, the rods in turn being assisted by cast-iron struts.

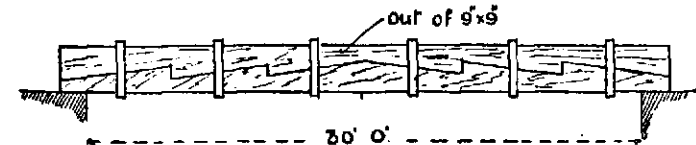


FIG. 323.

Their use is frequently resorted to in structures such as light road bridges and temporary work, but the practice has fallen into disfavour lately, partly owing to steel sections being so cheap, the unsatisfactory manner in which blacksmiths carry out the forgings, and finally to the unequal expansion set up between the timber and steel. A good example of a trussed wood beam occurs in figs. 327, 328, and 329. It would be suitable for a span of 30 ft., and is calculated as described under Beams. A fuller example will appear later on.

We now have to consider struts and pillars of timber when supporting beams. Regarding the joints between beam and pillar, called the bearing joints, two good forms occur in figs. 330, 331. These are designed in accordance with the theory of pillars and struts, fully described in that chapter, and are such that they more or less fix the pillar at the top end, and



FIG. 324.

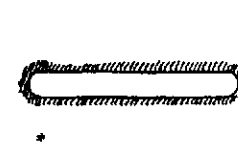


FIG. 325. — Machine-cut Mortise.

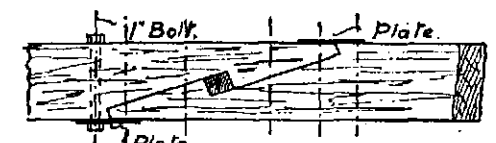


FIG. 326.

consequently make it much stronger, especially that shown in fig. 331. It is for such a reason that these bearing joints must *never* be made as in fig. 332, although the author has seen ignorant carpenters do so.

For temporary works the halved joint shown by fig. 333 is very satisfactory, but the double-halved joint shown in fig. 334 is more so, only it requires very careful fitting. When a post has suitable bracing, a common fished joint may be used, but when no such bracing is provided, fish plates must be provided on all four sides. The splayed scarf, fig. 326, can also be used for posts. In most heavy work, however, some form of bracing at the top of posts is provided for by means of struts. Two good forms are shown in figs. 335 and 336.

To find the required scantlings would only require a very simple stress diagram. We refer the reader to a previous chapter. In figs. 335 and 336, however, care must be taken that the joints shown in A are sufficiently set back from the ends of the beam, so as to prevent failure by detrusion, viz.

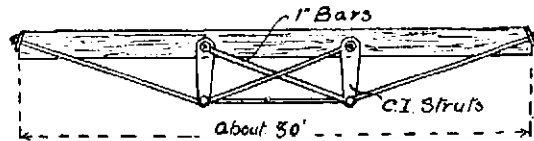


FIG. 327.

shearing off of the shaded area. The same conditions apply to joints at the feet of principal rafters and tie rods in roof trusses, and between the struts on king and queen posts. A more satisfactory way of making such joints, where practicable, is as shown in figs. 337 and 338. For fastening timbers, instead

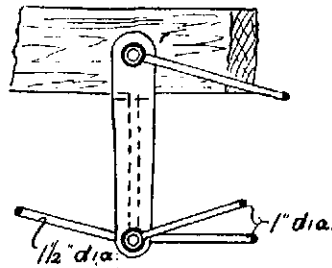


FIG. 328.

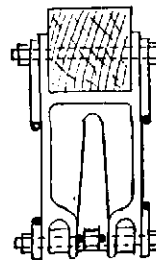


FIG. 329.

of nails, dogs, spikes, or bolts, we often use what are termed treenails. They should be made of best oak, having an ultimate resistance across the grain of 4000 lbs. per square inch. Their diameter should be $\frac{1}{3}$ of the thickness of the pieces joined. The use of iron or steel straps in carpentry is much in vogue;

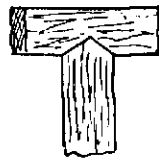


FIG. 330.

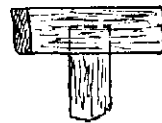


FIG. 331.

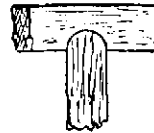


FIG. 332.

they are sometimes essential, but are only more or less satisfactory, because the timber, always liable to shrink, becomes loose in the straps. They are also liable to corrosion (especially in oak, due to the presence of gallic acid), and in all cases should undergo a process of preservation. A common and effective way is to heat the iron to the melting-point of lead (630° F.) and dip them in cold linseed oil. They may also be similarly dipped in coal-tar, or they

may be galvanised. The latter process fails, however, in sea air, or near sulphur fumes.

For some purposes it is necessary to bend timber slightly to an arched form. It is then called a *knee*. Its execution requires a special cramp. This cramp is shown in fig. 339. The strip of boiler plate goes right round.

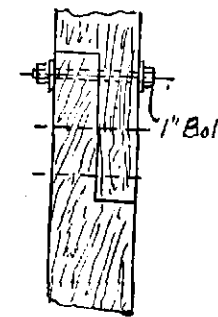


FIG. 333.

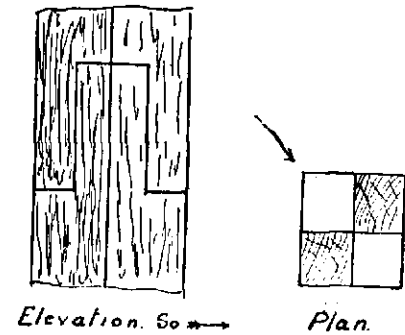


FIG. 334.

When beams are built up of timber placed side by side, they are called *built-up ribs*. They may be considered as strong as a solid rib of the same depth, and of a thickness less one piece. When, however, the layers are laid flat, it becomes a *laminated beam*. The several pieces are called the *laminæ*. They are weaker than solid ribs in the following way. A laminated beam of 5

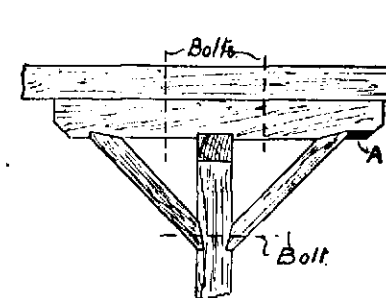


FIG. 335.

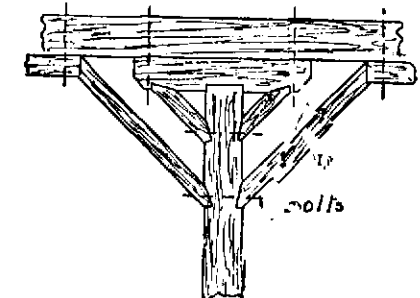


FIG. 336.

laminæ is only $\frac{1}{5}$ as strong as a solid one of the same dimensions. In most timber structures the tenacity or resistance to direct crushing should not be more than 1000 lbs. per sq. in. This in practice gives a factor of safety of 10. Assume it to be required to construct a timber bridge to carry a roadway over a river, canal, or other natural obstacle. First of all we decide upon the span = 8. Then say we place the beams at a distance B, centre to centre, and make the depth of beam = d. All measurement in feet.

Then to find the thickness of the beams we have the equation

$$T = \frac{BS^2}{1280d^2} \quad (225)$$

if it is a plank roadway we have decided to adopt. But if we are going to place a broken-stone roadway thereon, we have to modify our equation to

$$T = \frac{BS^2}{768d^2} \quad (226)$$

But we can also revert to the familiar formula,

$$W = \frac{2cbd^3}{L}$$

With such a system of girders it is usual to provide cross bracing, and wind pressure will also perhaps have to be considered. Bridge trusses in lattice



FIG. 337.

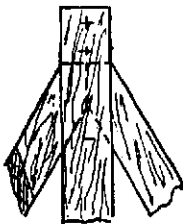


FIG. 338.

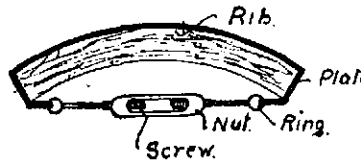


FIG. 339.

work will, of course, be treated in precisely the same way as steel ones, either by the methods under design of structures or by graphic statics, where we worked an example out in full. The planking may be laid diagonally with advantage, as by this method we get a certain amount of bracing power out of it: $\frac{1}{2}$ -in. space must be left between each plank for drainage purposes.

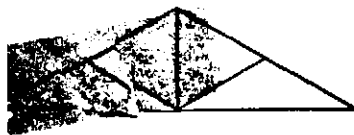


FIG. 340.



FIG. 341.

One of the most important applications of timber in construction is that of roof trusses. We are not, of course, concerned with small ones, such as occur in ordinary dwelling-houses, but a few examples of large trusses may be of advantage. It was also stated that graphic design was hardly applicable to wood roof trusses, so we shall discuss a few formulæ here, by means of which the correct scantlings may be arrived at.

The most important forms of frame or truss for roofs are—

The king post	fig. 340	} diagrams only,
„ queen post	„ 341	
„ mansard	„ 342	
„ hammer beam	„ 343	

and other variations of the above which occur principally in steel roofs. Some of the forms appeared in the chapter on Graphic Statics in diagrams.

Let us consider the truss as shown in fig. 344.

Given c = the half span.

K = the rise.

W = gross load over the whole truss, which includes the reaction due to the wind pressure. Then the thrust along the rafters (max.)

$$= T = \sqrt{\left(H^2 + \frac{W^2}{16}\right)} \quad (227)$$

and H = tension in the tie rod

$$= H = \frac{Wc}{4K} \quad (228)$$

considering the main truss.

The load at 1 is taken as $\frac{1}{2} W$, and the load at 6 is taken as $\frac{1}{4} W$.

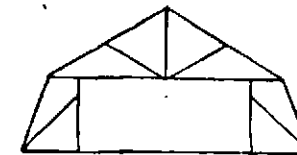


FIG. 342.

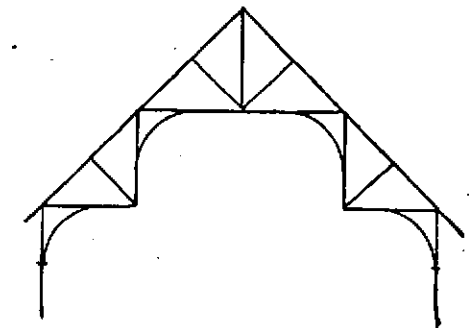


FIG. 343.

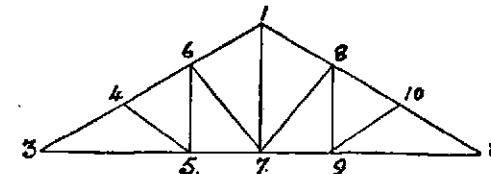


FIG. 344.

Now considering the secondary trusses,

If H_1 = horizontal stress in each of these,

T_1 = thrust on rafters between 6-3 and 8-2,

S_1 = thrust along 6-7 and 8-7,

$$H_1 = \frac{Wc}{12K} \quad (229)$$

$$T_1 = \sqrt{\left(H_1^2 + \frac{W^2}{144}\right)} \quad (230)$$

$$S_1 = \sqrt{H_1^2 + \frac{W^2}{36}} \quad (231)$$

Now the suspension rod 1-7 supports $\frac{1}{2} W$. But we before stated that the load on 1 = $\frac{1}{2} W$. But on 1 we support $\frac{1}{2} W$ directly, and $\frac{1}{2} + \frac{1}{2} = 1$. Coming now to the small trusses at the bottom, that is, 3, 4, 5, and 9, 10, 2, the points 4 and 10 each support $\frac{1}{4} W$.

Then if H_2 , T_2 , and S_2 , apply to these trusses, in the same way as the previous ones,

$$H_2 = \frac{Wc}{12K} \quad (232)$$

$$S_2 = \sqrt{\left(H_2^2 + \frac{W^2}{144}\right)} \quad (233)$$

and $T_2 = S_2$ in this case.

Now it remains to find the pull on 5-9. Its value is the sum of the pulls due to the primary truss and the larger of the secondary ones,

$$= H + H_1 \quad (234)$$

In the same way the pull between 5-3 and 9-2

$$= H + H_1 + H_2 \quad (235)$$

By a similar reasoning, roofs with rafters having more than 3 divisions

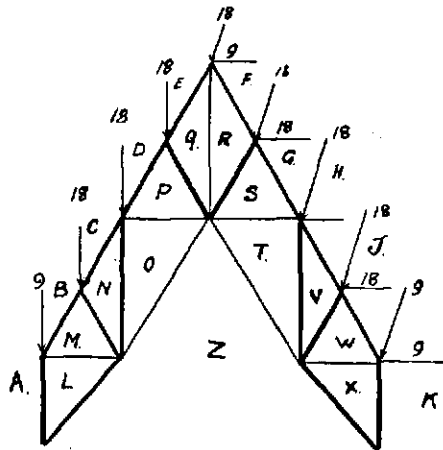


FIG. 345.—The figures denote loads in cwt.

can be successfully treated; but as timber roofs are not generally so constructed, we do not propose to deal with that here, while for steel roofs, graphic statics are most satisfactory. For a roof truss of the hammer beam type, the calculations necessary are of a somewhat tedious nature, and we can get the stresses set up in a fairly accurate and practical way by means of the following diagram, figs. 345 and 346. In roofs of this character it is not sufficient to take account of structural load only: owing to the high pitch, wind pressure must also be taken into consideration. Fig. 345 shows the frame diagram; and assuming a structural load of 28 lbs. per ft. super., and horizontal wind pressure of 42 lbs. per ft. super., the loads will be as marked, whence the stress diagram fig. 346 may be easily constructed. The curved members, fig. 347, should be calculated by the formula

$$\frac{W}{A} \pm \frac{M}{Z},$$

where W = direct stress in member in cwt., A = area in sq. in., M = bending moment in cwt.-inches = direct stress $\times$ versin of curve, Z = section modulus in inch-units. These stresses, when wholly compressive, should not exceed the safe working stress, taking the member as a strut with one end fixed and the other rounded.

Regarding the several advantages and disadvantages of the trusses referred to, the king and queen trusses are those usually employed in engineering structures such as power stations and like buildings, when a timber roof would

be considered better than a steel one. They would doubtless be open, that is, no ceiling joists, etc., and might be made of oak, and finished off well for the sake of appearance. There is not much to choose between them, except that the king post truss is not well suited for a span of more than 30 ft. In all cases the unsupported lengths of the common rafters should not be more than 8 ft., that is, struts must be provided. The purlins must come directly over these struts. The struts should also be as flat as possible, and have their feet resting as low as possible on the king or queen posts.

The ends of the tie beams must in no case be built into the walls, but lie in pockets specially provided for them; the tie beams should rest upon stone templets; they are not usually fixed very firmly. Regarding the pitch of roofs

they vary, usually for architectural reasons. For ordinary purposes 30° or $26\frac{1}{2}^\circ$ to the horizontal is generally adopted; the former has a rise = $\frac{1}{2}$ the length of the rafter, the latter = $\frac{1}{4}$ the span. The flatter the roof the heavier the slating (*vide* end of this chapter).

As regards the mansard roof, it is quite an architectural feature, and is only given here to point out three peculiarities, that is—

1. The idea of a great gain of space by its use is erroneous.

2. It is very inflammable, and should be allowed in cellings only under special circumstances.

3. It is an extravagant construction.

The hammer beam roof is also quite an architectural feature, but as its calculation affords rather an interesting stress diagram it is given here.

Roofs into which much iron or steel is combined with timber are known as composite roofs; the steel members would, of course, be applied mainly to the parts in tension. Cast-iron shoes and heads are used in conjunction with

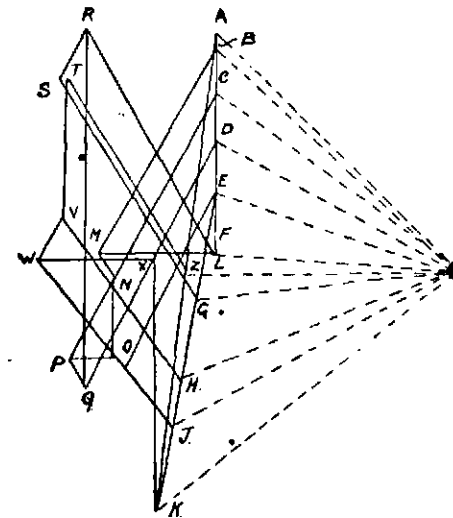


FIG. 346.

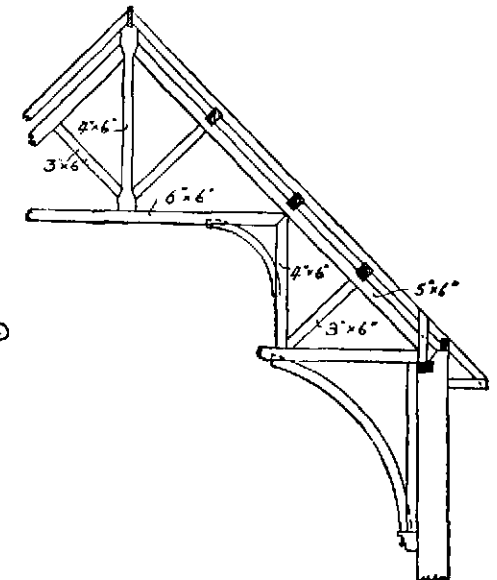


FIG. 347.

that of the engineer. At the same time, a book of this description would hardly be complete without a few of the more important and heavy details.

Our next important question is that of centering for arches.

The framework which supports an arch during its construction, when it

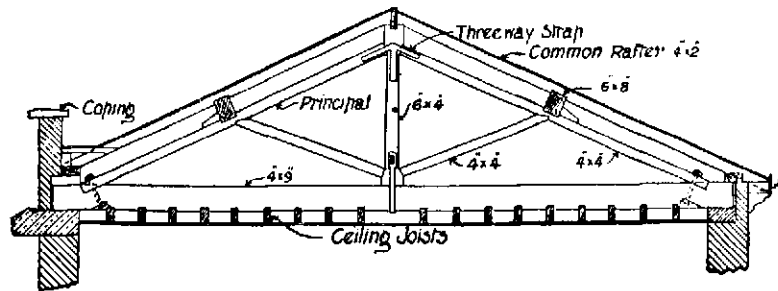


FIG. 355.

obviously has no supporting powers of its own, is called the centering. The designs are legion, from the curved lintel supporting the relieving arch of a door or window, to the structure which is to support some huge masonry arch of 200 ft. span. The latter detail would, in fact, be a design alone in itself, and would require plenty of engineering skill and forethought. In

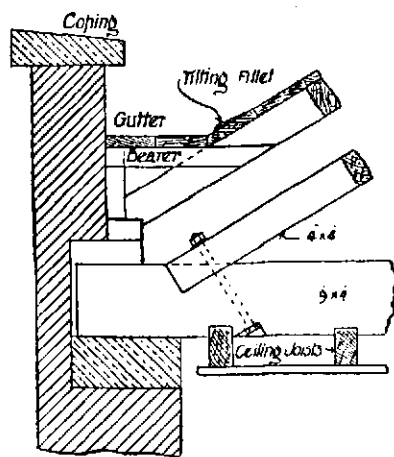


FIG. 355A.

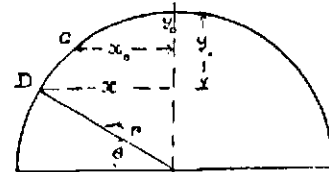


FIG. 356.

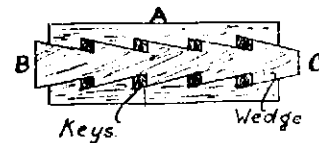


FIG. 357.

small spans for ordinary arches and for sewers and culverts the centering is supported on either side by posts to be provided; and if these occur in a river in which there is anything like a swift current, the process of erecting such a centering is often the most ticklish part of the work. Besides, they must, of course, be absolutely rigid.

Taking now w = weight in lbs. per lineal foot of the intrados (or soffit) of any arch. Take any point D in that soffit and join that point with the centre

fig. 356. Note the angle that it makes with the horizontal and call it θ . on the equation

$$P = w \cos \theta \quad (236)$$

will give us the *greatest* stress likely to come on to the laggings (the narrow strips of wood attached to the ribs which actually support the voussoirs). Referring to fig. 356 again, the normal pressure against the rib at the point D lineal foot of the intrados, using the notation on the figure,

$$= P = w \cos \theta - \frac{1}{r} \int_{y_0}^y w dy \quad (237)$$

being the radius of curvature.

Again, if we want to find the vertical load between C and D due to the pressure of the arch stones, in order to calculate the scantling for a post, its value is

$$Q = \int_x^s p dx \quad (238)$$

if we do not always support an arch on a series of posts—it is sometimes possible. Then we have to treat it as a girder, and if s = the half span of the girder, and we want to find the bending moment in the centre of the span,

$$M = Qs - \int_{x_0}^{x_1} p x dx \quad (239)$$

and, on any segment of the rib immediately under the laggings, and inclined to the horizontal θ degrees, we have the total transverse load on that piece

$$= Z = Q \sec \theta \quad (240)$$

The connection between the centering and the posts is made by means of *keys* or *striking plates*; it is by means of these that the arch is made to settle and take up its load. Regarding the period to do such preliminary loosening, the reader is referred to *Arches*. A special form of connection with fig. 373 is shown in fig. 357. The keys are first driven in, then the wedge, which should have been previously well greased, is tapped gently with a hammer; this will cause the centre to descend. In the actual design centerings there is nothing particularly abnormal. Experience and adaptability to circumstances are the principal factors. One of the most successful pieces of centering work (as well as the bridge itself, which we shall describe) was that of Mr Hartley, a Liverpool engineer, for the Grosvenor Bridge at Chester. It is stated, however, that Mr Hartley did not design the actual centre, but Mr Trubshaw, contractor. Four temporary piers had to be sunk in the river; on the top of each was a special cast-iron shoe, fig. 358, to receive the fan-like laggings; these framings were 6 in number, and were strengthened by

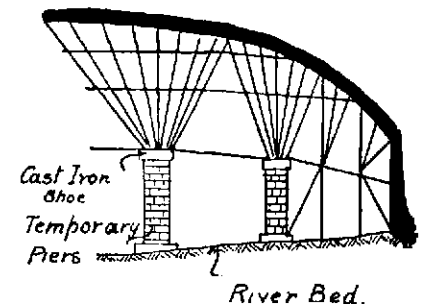


FIG. 358.—Diagram of Hartley's Centre.

such members; they are not very much now in vogue in England, because unequal expansion of the two materials (timber and steel) caused them to work loose and give unsatisfactory results, which, however, would be capable of correction in the case of open roofs more than others. In fig. 350 we give a fully dimensioned example of a large wood roof truss on a king-post principle



FIG. 348.

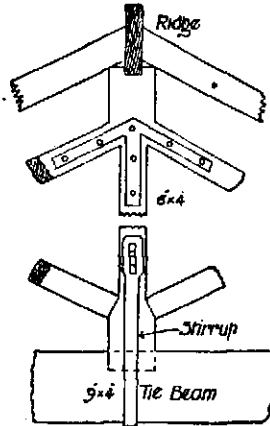


FIG. 349.

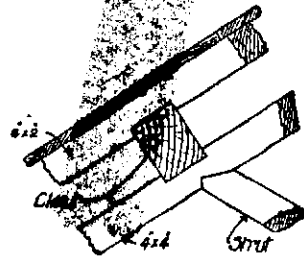


FIG. 350.

which might occur any day in practice, suitable for a span of about 20 feet. The ceiling joists are shown to give the reader an idea how they would occur if required. At the top of the king post is an arrow-shaped strap used for fastening the struts; though much used, they are unfortunately liable to work loose. At the bottom of the king post is a special form of joint called—

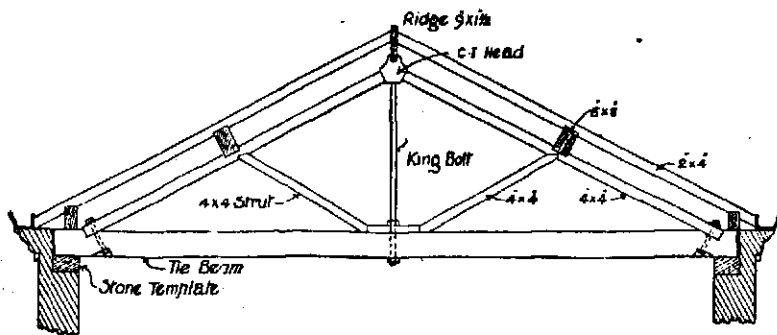


FIG. 351.

The Jib and Cotter Joint.—It is shown in detail in figs. 348 and 349. Its object is to take up the slackness which invariably ensues between the king post and the tie beam due to shrinkage of the former. For this purpose it is very effective, but it wants a little care in design, the two points of main importance being the two clearance spaces, one between the top jib and the king post, and the other between the strap and the bottom jib. Without these the joint cannot be tightened up. The tightening up is of course effected by driving

home the cottars. This joint, in one or other form, is used much in engineering construction. The jibs must be well greased. Another detail of the same roof is shown in fig. 350. Fig. 351 shows a similar type of roof, in which, in place of the king post, there is a king rod of iron or steel, tightened up by a nut at

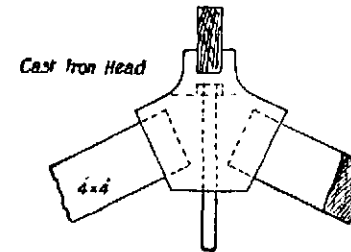


FIG. 352.

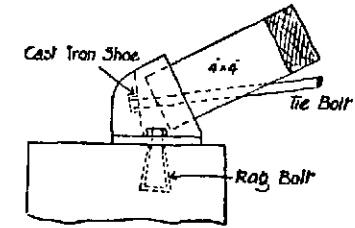


FIG. 353.

the bottom; there is also a cast-iron head, shown in detail in fig. 352; we may also have a wrought-iron or steel tie rod, which might be an advantage, in which case we would provide a cast-iron shoe at the foot of the principal rafter, shown in fig. 353. We do not show a queen-post roof on a large scale, as its construction is somewhat similar to that of the king-post roof, and the diagram in fig. 341 will explain it. But a detail is shown of the queen post and its fastenings in fig. 354, fastened by means of a 3-way joint at the top, and a jib and cotter engagement at the bottom, similar in every detail to the one before described. These roofs would be suitable for a factory or other such engineering structure, and in such places where the roof would be open and the joints could be periodically examined, tightened up, the ironwork painted. Nowadays, however, the design of large wood trusses has largely fallen out of the engineer's hands and given way to steel trusses, which in such cases are cheaper to execute and more efficient. They are also to a great extent undamaged by fire or great

We shall return to them under the heading of Steel Construction. In fig. 355A we give another detail of a wood roof; it is applicable to either the king- or queen-post roof, and is for the purpose of providing a parallel gutter when the main walls terminate in a parapet. It will of course add to the expense, and is usually confined to large buildings. It is very prevalent in London in the large houses; it is of course necessary to tie the ends of the tie beams in pockets. What we have said in reference to timber roof trusses is, of course, by no means intended to be exhaustive, because their design belongs to the profession of the architect rather than

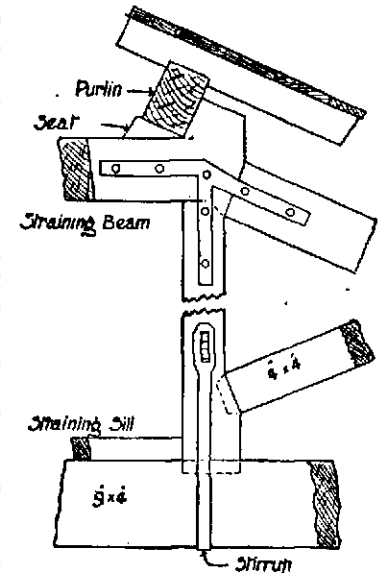


FIG. 354.

cross-pieces (vide *Trans. Civ. Engs.*, vol. i. p. 207). There was a separate wedge under each arch stone, so that any portion could be relieved at pleasure. We will now proceed to give a few practical examples.

A common form which would be used for a semicircular arch for any

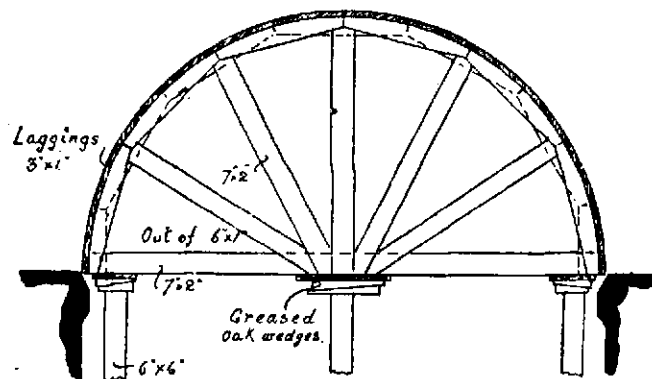


FIG. 359.

thing up to 30-ft. span, when intermediate supports would be allowed, is shown in fig. 359, set up ready for building the arch upon. The scantlings are marked thereon. In 360 is a section of the same through the centre. Fig. 361 shows a much larger piece of work for a span of 50 ft. to 60 ft. or

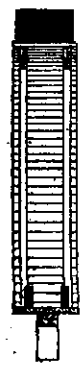


FIG. 360.

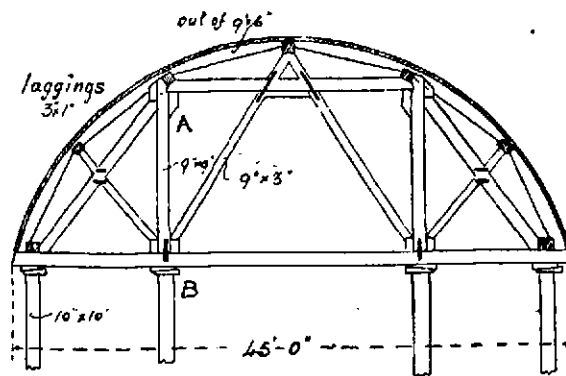


FIG. 361.

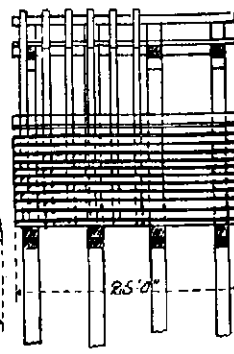
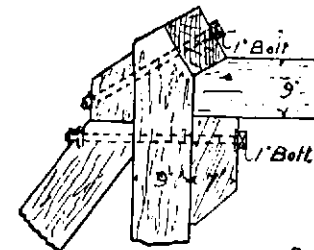


FIG. 362.

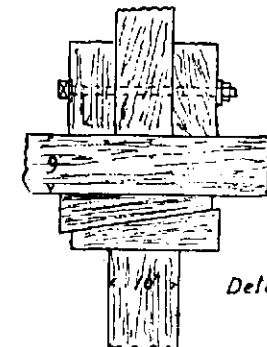
less. The several parts of the drawing should be carefully studied. A part cross section and end elevation occurs in fig. 362, while 363 shows a detail at A and 364 a detail at B. This is a good type of centre which would find numerous applications, and is at the same time fairly economical of material.

If, however, as we stated, the arch was to be constructed over an obstacle, under which condition we could not have intermediate supports, our centre would have to be of a different design. If of a large span, it must rest on a

specially designed trussed girder, which must in every way be calculated as any other such girder.* In fig. 365 we have such a centre, over a short span 27-30 ft. only, in which the scantlings only assume moderate proportions. A cross section is shown in fig. 366. A type of centre which requires much forethought in its execution (as does that of the arch itself, in fact) is the one used to support a groined vaulting,



Detail at A.



Detail at B.

FIG. 363.

FIG. 364.

It is principally an architectural feature of churches, and may at first seem foreign here, but it seems to be the practice now to construct the roofs of closed service reservoirs in this way (e.g. the Honor Oak reservoir for the London Water Board, 1909), where it is considered advisable to eliminate steel from the construction. A plan of such a vault having solid walls on two

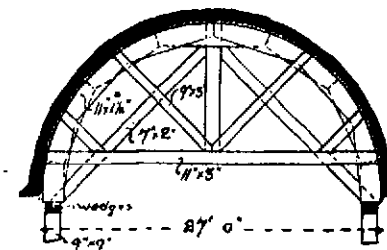


FIG. 365.

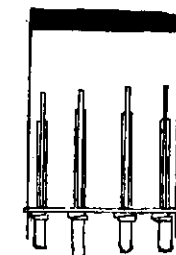


FIG. 366.

sides is shown in fig. 367. Here we have the main rib No. 1 going from corner to corner, shown in detail in fig. 371. Abutting on this, at right angles to it, are two half centres of the same profile, shown in fig. 370; No. 4 is the pointed centre shown in detail in fig. 368, while No. 3 are the intermediate centres shown in fig. 369. The construction of all these is shown in full detail. There are supporting posts as shown. To get the profile of such centres requires, however, a careful graphical construction. This, which should be done to a large scale on paper, is shown in fig. 372. We proceed as follows:—

1. Draw a plan of the diagonals.
2. On each end draw a section of the proposed arch.

* Vide Equation 239.

3. Draw equidistant (dotted lines) lines to meet the curves.

4. Where these lines intersect the curve, draw perpendiculars to intersect the diagonals. Now draw *ef* parallel to the diagonals, and the dotted lines the same distance apart as before.

Then, from the points where these lines cut, the lines drawn perpendicular to *bd* at the intersecting points will give the actual points of the required curves.

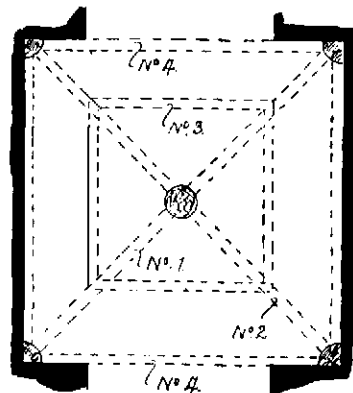
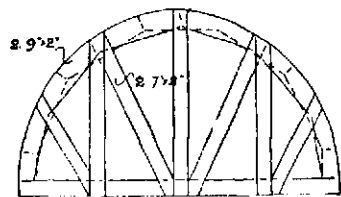


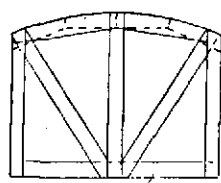
FIG. 367.

and is cheap to execute. Two other suitable forms of centering (diagrams only) appear in figs. 375 and 376, while one for a flat elliptical arch in detail is shown in fig. 377.



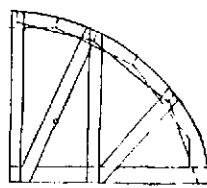
N° 4.

FIG. 368.



N° 3

FIG. 369.



N° 2

FIG. 370.

We shall now proceed to turn our attention to timber bridges. In Chapter XII. we worked out fully a timber bridge for a 100-ft. span under rather difficult circumstances and obtained the scantlings. The example, though particularly suited to the chapter in question, is not a very good practical example of such a bridge truss. Therefore we shall show in detail a truss which would fill the conditions enumerated in a very satisfactory manner. A fully dimensioned elevation and section occur in figs. 378 and 379; cast-iron sockets would be provided at A.

For a common roadway, we stated before in Equations 225 and 226 some particulars where trussed beams would not be employed. Such a form of bridge would only be suitable over a small span, unless the material was used to a great disadvantage. A better way to carry out such work is to employ trussed girders and support the bridge either on the top or bottom booms.

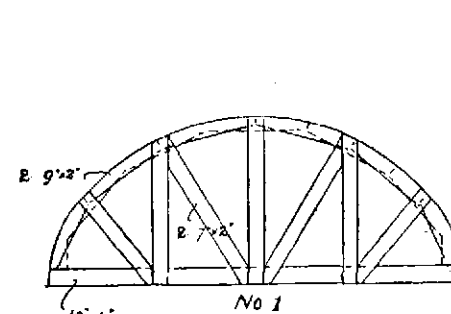


FIG. 371.

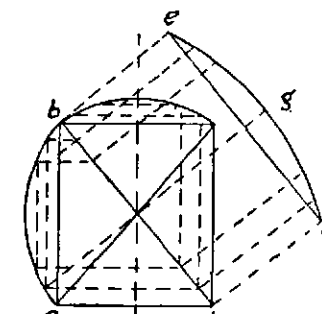


FIG. 372.

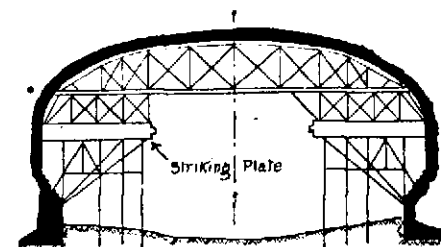


FIG. 373 (see Fig. 367).



FIG. 375.

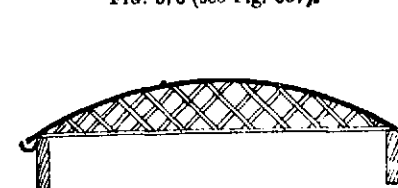


FIG. 374.—Belfast Truss.

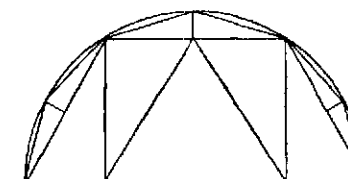


FIG. 376.

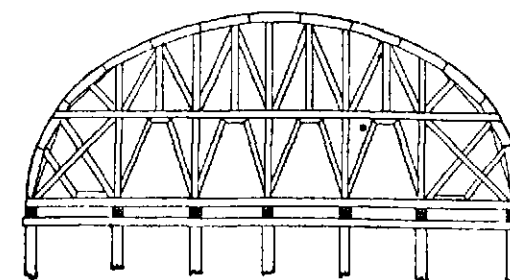


FIG. 377.

A very suitable construction for a 25-ft. span and ordinary road traffic is shown fully dimensioned in figs. 380 and 381.

The drawings themselves are quite self-explanatory, and the stresses and

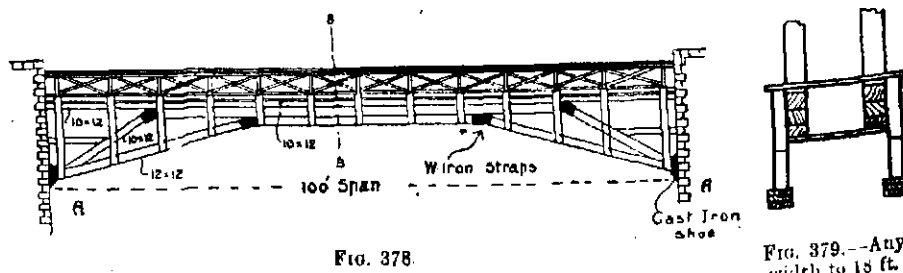


FIG. 378.

FIG. 379.—Any width to 15 ft.

scantlings can be easily worked out by diagrams, or by the methods explained in the chapter on Design of Structures, when the maximum loads and span are known.

Laminated rib construction, already referred to, has been used to some

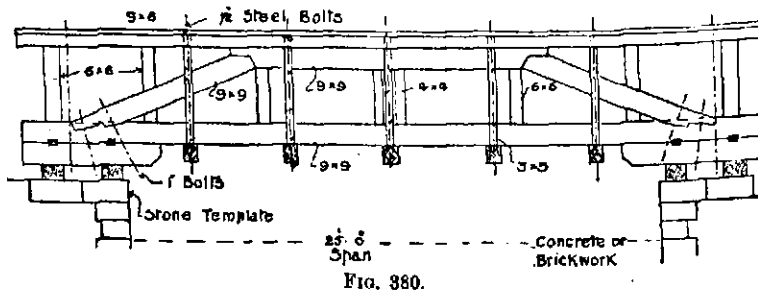


FIG. 380.

success in light bridge construction in timber, especially in America. A design for such a bridge is given in figs. 382, 383, and 384. There is a central roadway and two footways, one on each side. The road is paved with wood blocks, while ordinary planking serves the footways. The rib, which consists of 6 laminæ, springs from cast-iron sockets, which in turn rest upon stone templates. The abutments take the whole thrust of the arch, and must be calculated to withstand the same. We discussed the calculations under Arched Ribs. The general construction is made

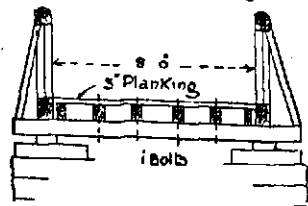
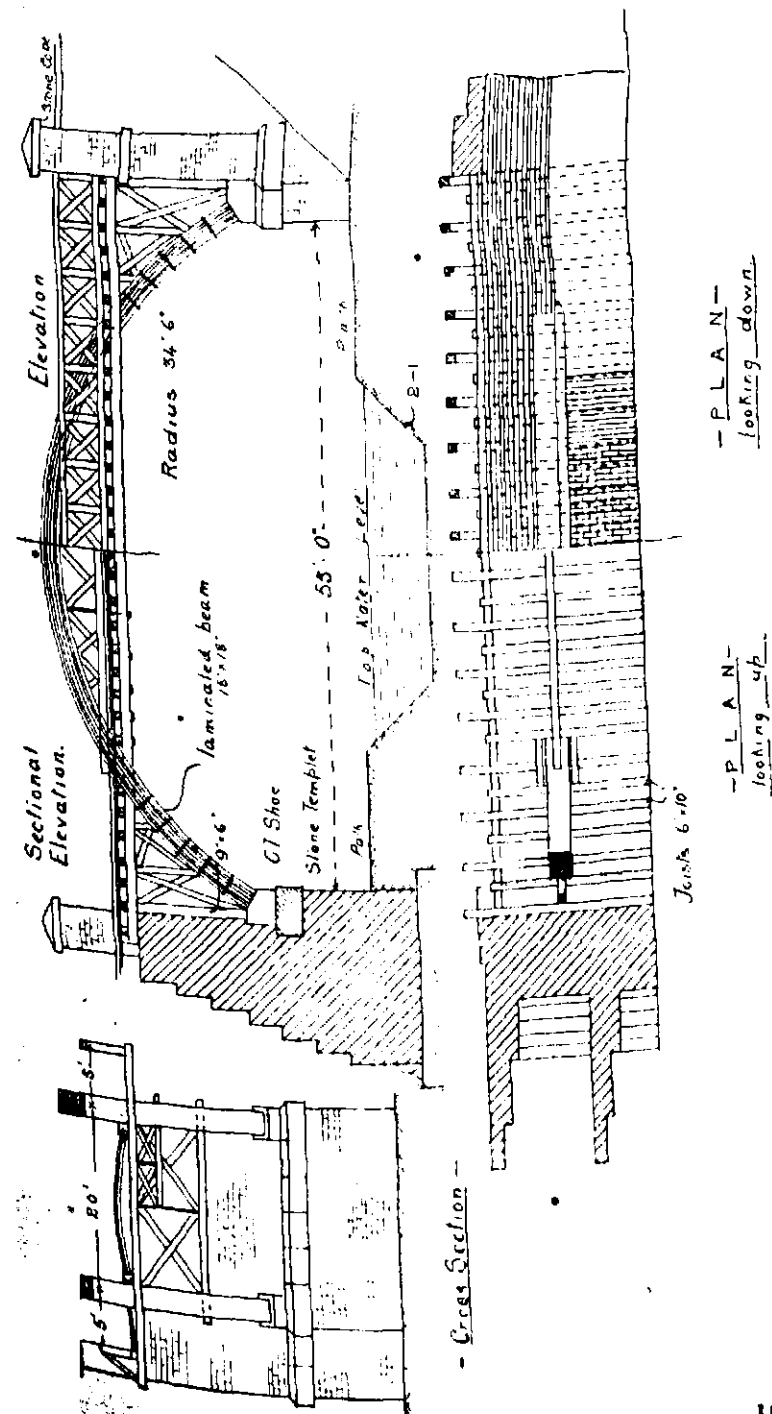


FIG. 381.

fairly clear by aid of the drawings.

The last important feature in timber construction which confronts the civil engineer is that of *shoring up* dangerous buildings, or those rendered unsafe during structural alterations to the neighbouring ones. When such timbering is applied to supporting the walls of buildings during alterations, it is called *needling*. A typical example of the latter process is shown in p. 385. In this case it was required to cut out the lower part of the main



FIGS. 382, 383, and 384.

wall of the building in question to admit a rolled steel joist, a proceeding very common when altering buildings for shop windows, etc.

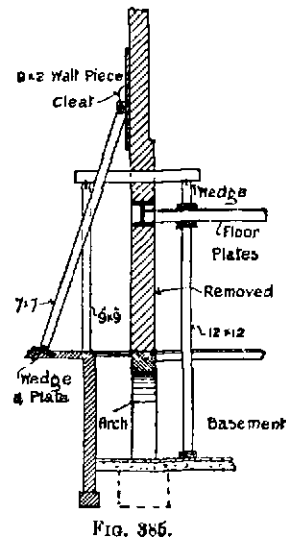


Fig. 385.

First fix up the wall-pieces and fix the needles into holes that have been previously prepared; these needles are mortised through the wall-piece into the wall, in line with the floor joists of the building. The cleat that is housed

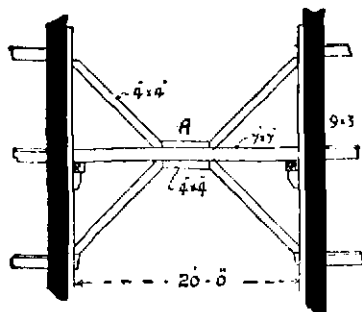


Fig. 386.

in underneath strengthens the projecting end of the needle; upper and lower cleats for taking the struts are housed in about $\frac{3}{4}$ in. and nailed. These cleats are also fixed in the line of thrust against the upper and lower floor when this method is convenient, as shown in fig. 387. Next fit in the lower struts at a slope of about 45° , and the straining-piece should be a good tight fit. Then fix the upper struts. Oak folding wedges are driven in at the end of the horizontal shore against the wall-piece, and finally the wedges are driven in at A* (fig. 386), which has the effect of tightening the four struts

* Between the struts and horizontal straining-pieces.

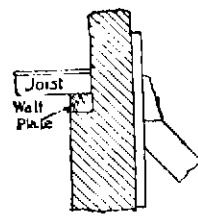


Fig. 387.

Fig. 388 shows a flying shore that is used when the house that is to be supported is much higher than the house that is used as an abutment. A raking shore and a rider are abutted against a wall-plate on the building opposite. The struts are fixed with dogs or spiked across the two shores on to the horizontal shore, where packing-pieces are fitted between so as to make a fixture for the struts. The horizontal shore is wedged, as also is the rider or top shore. The whole structure is well fixed with iron dogs so as to prevent movement.

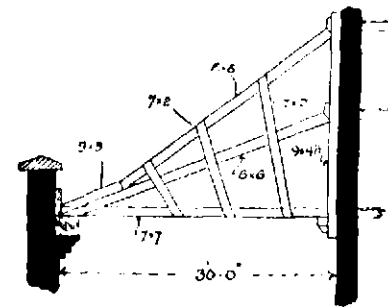


Fig. 388.

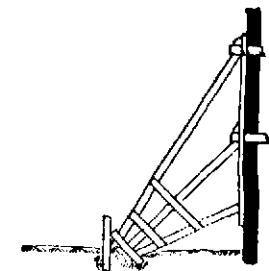


Fig. 389.

Another example of a shore is shown in fig. 389. Before concluding the present chapter it would be well to give another example of a timber lattice girder when the roadway would be constructed on the top boom. Such a case suited to a span of 30 ft. is shown dimensioned in fig. 390. It would, however, without much modification, be quite possible to carry the load on the bottom boom.

The drawings for a centre, designed to suit some special circumstances, are shown in fig. 390a.

For a bridge, the span of which is from 60 ft. to 100 ft., the arch is elliptical. The struts that are marked A are birds-mouthed and fixed with dogs, and the struts that are marked B and the pieces marked C are bolted in pairs on the outside. The pieces marked D are stretchers, which are carried across to the next rib, which should not be more than 9 ft. apart. The centre is raised by a continuous wedge, which should be well greased.

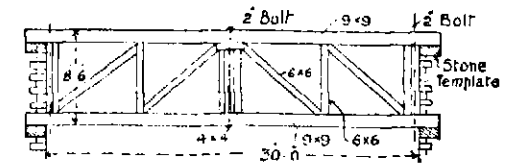


Fig. 390.

The illustration shows a method of throwing the weight on to the foundation offsets.

We also have the opportunity of presenting to the reader an example of constructional carpentry very likely to confront him—that is, a staging to carry a derrick crane. These cranes are now much used by municipal and constructional engineers, owing to the ease with which large weights can be easily manipulated. The drawing shows the elements of the design for such a piece of work, and the scantlings given would suit a crane up to $3\frac{1}{2}$ tons capacity. Sometimes these stagings are erected to a great height, and the

jibs of the cranes thereon enormous. It is very important that sufficient ballast be provided at A to prevent any chance of the crane toppling over, due to load. The principle of moments can easily be applied to such reasoning. At the same time the actual weight of the crane must be taken into

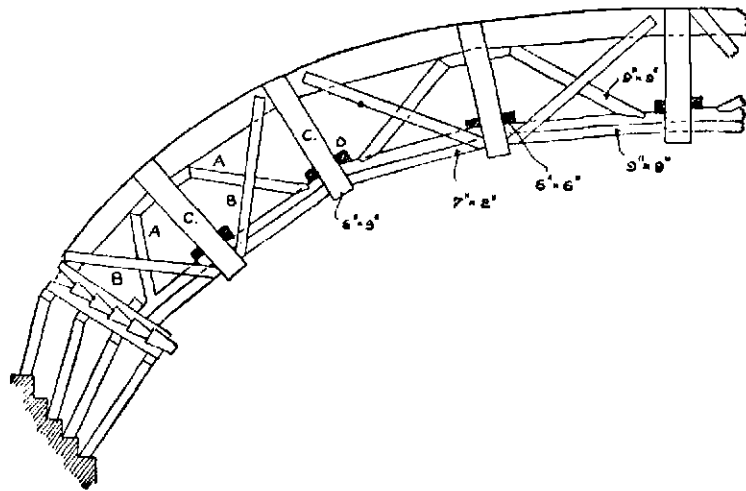


FIG. 390A.

account when designing. The crane may be steam, electrically or even hydraulic driven, the second named being the most adaptable (see fig. 390B).

* **The Pitch of Roofs.**—The pitch of roofs should, within certain limits, which are defined by the material that forms the protection against weather, be made to conform to and harmonise with the general design of the buildings

TABLE I.

Kind of Covering.	Inclination.	Proportion of Rise to Span.	Weight in Pounds on each Square of Roofing
Asphalted felt	18 26	$\frac{1}{2}$	30-40
Copper	3 50	$\frac{1}{10}$	80-120
Corrugated iron (16 B.W.G.)	4 0	$\frac{1}{20}$	350
Lead	3 50	$\frac{1}{20}$	550-850
Slates (large)	22 0	$\frac{1}{4}$	900-1100
„ (ordinary)	26 30	$\frac{1}{3}$	650-800
„ (small)	30 0	$\frac{1}{2}$	450-650
Thatch (straw)	45 0	$\frac{3}{4}$	650
Tiles (plain)	45 0	$\frac{1}{2}$	1600-2100
„ (pan)	24 0	$\frac{1}{3}$	1200
Zinc	4 0	$\frac{1}{20}$	150
Boarding, $\frac{3}{4}$ -in.	26 30	$\frac{1}{3}$	250
„ 1-in.	26 30	$\frac{1}{3}$	350
Pressure of wind	...	...	2500-5000
„ „ snow	...	...	500

* *Building World.*

that are covered. The surroundings and the locality, the shape and size of the building, together with the purpose for which it is to be used, naturally have to be taken into account in determining the most suitable form of roof; and to these must be added other factors, such as economy of cost, facilities

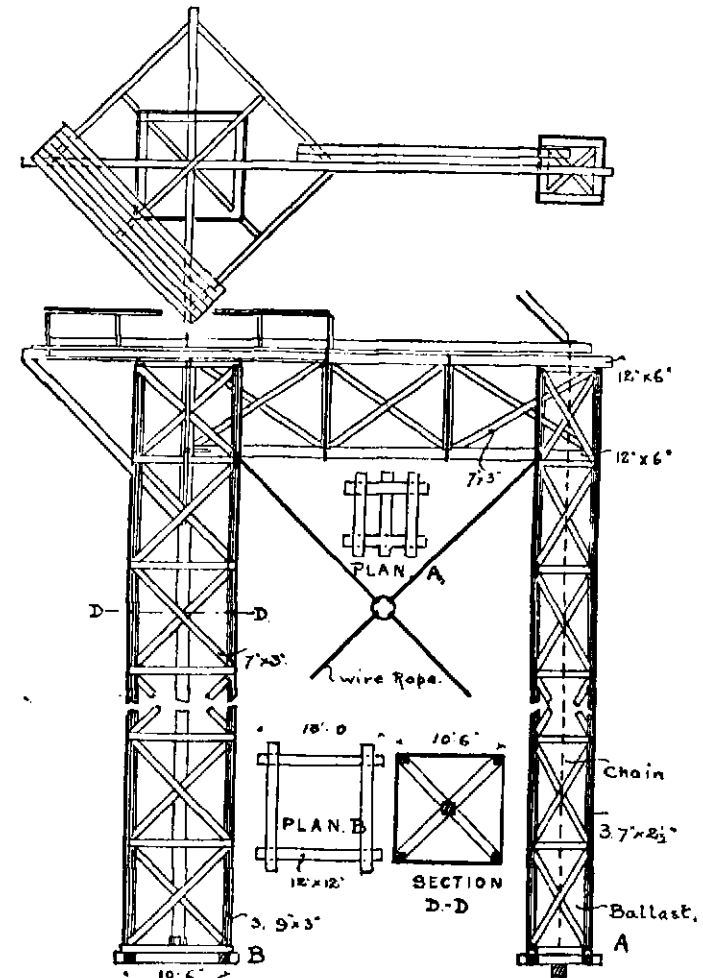


FIG. 390B.

for obtaining materials, relative weights when compared with other materials, and general durability. The table that is given above (Table I.) shows the kind of covering, together with the minimum inclination at which it may be used, and the proportion that the rise bears to the span in any ordinary couple roof with ridge, together with the approximate weight on each square of roof surface. A "square" is equal to a surface 10 ft. long by 10 ft. wide, that is 100 sq. ft.

This table has been compiled chiefly from Hurst's and Tredgold's figures. Metals like copper and zinc, which can be used in thin sheets of comparatively large size, so that few joints are needed, can be laid at a very small inclination, and weigh but little when compared with other forms of roof covering, such as tiles or slates. This lightness gives the advantage of allowing small scantlings to be used for roof members, and the small inclination gives shorter rafters, and consequently a smaller amount of roof covering, in addition to the smaller amount of timber required for supporting it. So that a light covering, if it can be laid in large sheets with weather-tight joints, is not only economical, as requiring a smaller amount of covering material, but also allows of a lighter and cheaper form of truss.

In the case of tiles or slates, the amount of lap that is allowed affects the weight that is added to the roof; but, of course, as the lap increases, so do the weathering properties of this particular roof covering. A roof that is covered with small slabs of material like slates or tiles has many openings through which rain and wind may find an entrance, and such materials have to be laid at a greater inclination, so that the rain may quickly run off. Also, the steeper the inclination, the less chance has the wind of blowing under the covering and rendering it loose. But, as already shown, the steeper the inclination, the greater is the length of the roof from the eaves to the ridge, and the greater is the weight that is put on the trusses and walls. Tiles, being more porous than slates, have to be laid at a greater inclination, and, as will be seen from the tables, they form a much more weighty roof. The pleasing colour of plain tiles has, however, a charm that common blue slates do not possess, and the extra expense that is necessary in making a stronger roof is well recompensed by the better appearance of a house so covered, especially if it stands amidst harmonious surroundings.

The proportion of rise to span is obtained by dividing the rise by the span, so that if a roof had a span of 24 ft. and a rise of 12 ft., the proportion would be one-half. Therefore, in order to use the set of figures under this heading, it is only necessary to know the span in order that the rise and a suitable inclination can be found. Thus, in order to design a roof for an ordinary building of 30-ft. span covered with slates, it would only be necessary to make the rise equal to one-fourth of 30 ft., that is, 7 ft. 6 in. To the calculation made by the use of the last column in Table I. an allowance must be made for wind pressure to the extent shown. The steeper the roof, the greater will be the pressure on each square of surface, and of course the pressure also varies according to the speed of the wind that is blowing.

TABLE II.—WEIGHT OF ROOF TRUSSES.

	Span.	Weight in Pounds.
	Ft.	
King post	20	390
"	30	1020
Queen post	40	1460
"	50	2230
"	60	3640

In Table II. (see above) is shown the weight of roof trusses, taken from Hurst.

These figures only give the approximate weight, and are useful as showing at a glance the weight that is put on the walls by each truss. In order to get a more exact computation, the number of cubic feet of timber in each truss must be calculated and multiplied by the weight of the timber per cubic foot, and to the amount so obtained must be added the total weight of all iron straps, bolts, or other fastenings that are used. Should the trusses carry a ceiling, the weight of the portion that is carried by each truss has to be added before the total amount carried by the walls and the trusses themselves can be correctly estimated. In calculating the weight that is borne by any piece of construction, its own weight is an important factor, especially when a heavy material like iron or steel is adopted.

Since preparing the bulk of this chapter I have been enabled, by the courtesy of the architects, to whom I am greatly indebted, to present to the reader a fine modern example of constructional carpentry in the oak foot-bridge at Guildford.

In this age of cement and iron it is pleasant to find an example of a bridge built in honest English oak. The new footbridge over the Wey at Guildford, which was formally opened by the mayor and corporation on August 25, 1909, is a picturesque structure, as well as a substantial one. The bridge, which cost about £400, was paid for by public subscription, and the subscribers and the inhabitants generally are to be congratulated upon having secured a structure which is appropriate to its purpose and its situation, and is in every way a desirable addition to their beautiful and interesting town.

The design of this bridge, which was chosen from among more than seventy submitted in competition, was modelled on the design of Palladio's bridge over the river Cismona, between Italy and Germany.

The clear span of the bridge over the river Wey and towing-path is 63 ft. 6 in., and over all the bridge is 78 ft. in length, and is 5 ft. wide in the clear.

At the west end the bridge is approached by a staircase, so as to leave a headway of 9 ft. to the towing-path, and at the east end the bridge lands the pedestrian on the Shalford-road level.

The bridge is constructed entirely of English oak, framed together and secured with wrought-iron straps and bolts.

The dimensions of the principal timbers are: Tie beams (each in three lengths), principal braces, queen posts, and straining-beam, 12 in. by 9 in., and the cross beams carrying the floor, 9 in. by 9 in.

The bridge is supported at each end on four 12-in. by 12-in. oak posts braced together, on sole-pieces resting on cement concrete foundations. The detailed drawing, reproduced on the folding plate, will make the methods of construction clear to practical readers.

The architects are Messrs Clements & Moon, of Guildford, and the contractors, Messrs R. Wood & Son, of Guildford. Messrs Henry Adams and Son, of 60 Queen Victoria Street, E.C., acted as consulting engineers.

CHAPTER XVIII.

THE CONSTRUCTION AND MAINTENANCE OF ROADS.

Road Materials.—The making and repair of roads are very important parts of a municipal engineer's business; in fact, county engineers devote most of their time to overseeing and looking after the upkeep of the roads under their charge.

During the latter part of the nineteenth century the roads, which together with the canals had been the sole means of inland communication, fell of necessity into much disuse, and consequent neglect. During the last ten years, however, the advent of the motor car has changed all this, and now the question of road maintenance (not so much construction) is a very vexed one, especially as most of the roads in outlying country districts are totally (or were so at least before the profession took up the matter) inadequate to deal with the fast and heavy traffic of motor cars. Nothing is more detrimental to the reputation of a man in a public position than bad roads.

Road "metal," as the stone used in roadmaking is called, must be viewed from several standpoints; and when the surveyor is called upon to discriminate between different classes of road stone, he must carefully regard the following considerations, viz.—

1. Cost
2. Durability } these are very important.
3. Cleanliness, that is freedom from dirt, clay loam, soil, etc.
4. Weathering properties.
5. Uniformity of wear. A road which will wear unevenly is very undesirable, as it becomes pitted and difficult to repair.
6. Ease of traction.
7. Ease of foothold for horses.

Therefore, from the foregoing conditions we conclude that the essential properties of a good road stone are—

1. *Hardness.*
2. *Durability.*
3. *Toughness.*
4. *Binding properties.*
5. *Maintenance of a rough surface.*

This latter condition is only essential in sett paving stones which would wear to a smooth and slippery surface; such as hard granite would be unsuitable. The softer granites are always used for this purpose.

Now, before going on to describe the composition of the various kinds of road stone, it is essential that the reader be first well acquainted with the several minerals which go to make up a road stone. First of all we have—

1. *Biotite.* It is a mineral of the mica group, but is black in colour and contains some iron and manganese. It is soft, has a perfect cleavage, and will break up into thin, smooth scales.

2. *Apatite* is a natural phosphate of crystalline rocks. It gives off much phosphoric acid, and it is upon the acid that all vegetable life is dependent. It occurs principally as phosphate of lime, and is found in hexagonal prisms. It is soluble in dilute nitric acid. Its colour is principally sea-green, and is found in small, transparent, long-pointed crystals, which are invisible to the naked eye. It also goes under the name of "asparagus" stone.

3. *Hornblende* is a black or dark-green lustrous mineral. It will decompose with the formation of chloride, and is very like augite in appearance, but can easily be distinguished by its natural angle of cleavage of 124 degrees.

4. *Orthoclase* is potash felspar. It is sometimes combined with soda, and occurs in white, pink, or grey crystals. Decomposition is liable to set in, yielding kaolin and quartz.

5. *Kaolin* is china clay, and is found to a great extent in Cornwall. It is a hydrated silicate of alumina and a pure white powder. It is principally composed of granite in a state of decomposition. By the use of strong H_2SO_4 we can make alum from it.

6. *Olivine* is a ferrous and magnesian orthosilicate, and occurs in rhombic prisms of an olive-green colour. It will readily weather to serpentine.

7. *Muscovite* is potash mica. It is transparent and flexible, and only another name for mica.

8. *Pyroxene* is composed of silica, iron, and oxide of alumina. Its colour will vary; it does in fact range from white to dark green, or even black. The crystals may be rhombic or monoclinic.

9. *Pyrites* is disulphide of iron. It occurs usually in cubes of a brassy-looking colour. It is very brittle. Burnt in air, it will yield sulphur dioxide gas. When it occurs in an isometric form it gets the name of marcasite.

10. *Plagioclase* is triclinic felspar. Usually occurs in clear transparent crystals. It will decompose frequently into china clay (= kaolin).

11. *Quartz* is a very important mineral in the composition of road stones. Being crystallised silica, it is the principal ingredient of sandstone. It is a very hard mineral, being even harder than steel, but is very brittle. Its appearance is smooth and glassy. In its colourless form it occurs as rock crystal. What is known as quartzite is a white, grey, or yellowish rock, chiefly composed of quartz.

12. *Felspar* is a silicate of alumina. It may contain potash, soda, or lime, and usually of a white or grey colour. It will decompose upon exposure to the atmosphere, and lose its glassy appearance thereby. It occurs in two main groups, monoclinic and triclinic.

13. *Felsite* is a fine-grained igneous rock of the acid variety.

14. *Augite* occurs in all igneous rocks, especially basalt. It is usually in black monoclinic crystals of short and stout shape. It has the property of decomposing into "greenstones" such as serpentine.

The Properties of Road Stones in general.—The stones used by surveyors for road purposes are usually natural inorganic rocks. As stated, all stones must be hard, tough, and durable. Many stones which on examination would have a naturally good appearance in the quarry, would, when exposed to the air, become rotten. This would be due to various chemical actions, set up in a great measure by horse manure. It may here be stated that the fact of

being able to judge with fair accuracy a good road stone by sight is a valuable acquisition of the road surveyor. It is, of course, only gained by practical experience. To attempt the question here would be ridiculous; all we can say is, the practice should be cultivated.

One of the best rocks for use in macadam roads is *granite*. It is what is called a plutonic igneous rock, and was formed under great heat and pressure. It is holocrystalline and an acid rock. [What is here termed an acid rock is a rock which is known to contain more than 66% of silica, and the more silica a stone contains the smaller will be its specific gravity.] The average composition of granite is

36% of felspar	}
11% of mica	
46% of quartz	

The granite from County Donegal has 65% of felspar. Hornblende is a valuable property of granite. It is, however, liable to yield to atmospheric influences and become clayey. Magnetite and pyrites are detrimental to its quality because they are oxidisers. When granite is used for road metal it must have a fine grain, and it is the possession of this latter property in a marked degree which has made the Guernsey granite so much favoured, especially in the south of England. The Killiney granite, so much valued in Ireland in the Dublin district, is what is called a *foliated* granite. Most granite will absorb about 1% of its weight in water. When it occurs in bosses, which it usually does, the stone at the top will probably be rotten and unfit for use. A special form of granite is found in Cornwall. It has special qualities, one of which is, it decomposes into kaolin. It is supposed to have been formed under considerably less pressure than ordinary granite. It is only found in small isolated masses, and is called *elvan*. It will prove a good road stone.

Before we proceed further it will be necessary to explain four common terms in reference to road stones. They are, *porphyrite*, *porphyry*, *decomposition*, and *alteration*.

When a rock has more soda lime than potash lime it is called a *porphyrite*.

When the conditions are reversed it is called a *porphyry*.

When a rock breaks down, due to chemical action, "decomposition" has set in.

When the structure is altered mechanically, "alteration" is what has taken place. Such alteration often tends to make a rock more compact.

Hornblende sometimes contains felspar. When so, the felspar is usually crystalline; but when quartz is present, it is simply mechanically interlocked; but this interlocking may reach such a pitch that the quartz is actually intergrown. An example of this is what is called *graphic* granite.

Orientation of a rock is its liability to split (e.g. slates and York stone for paving purposes). The term Basalt is used as a name for most igneous rocks having a very dark colour; they are usually low in silica, and are called *basic* rocks; this is in contradistinction to acid rocks containing more silica, they are naturally heavy; metallic oxides are also present in a marked degree; they make very good setts because they do not wear smooth. As road metal (as broken stone is called) they are also useful. The structure is usually compact. For making setts a specially adaptable stone comes from Penmaenmawr in North Wales; they are much used in Manchester, Liverpool, and Dublin, where they appear to have given satisfactory results. The Penmaenmawr stone is itself a cross between a diorite and basic diabase, principally

composed of felspar of a dark greenish grey colour and having a medium grain. Whinstone produces another favourite road metal; it is what is termed a *Dyke* rock, and comes principally from Cleveland Dyke and Whinsill. It will contain usually an average of 60% of silica. Its specific gravity is 2.77; it has the peculiar property that it is liable to slight decomposition, and if in this state a piece of the stone is placed in dilute nitric acid it will effervesce, and is very often mistaken for limestone on this account. It is, of course, a basic rock, and usually has very little quartz present.

From Porthweli in Wales we get a special stone from the Gimblet Rock. It is a very good road stone, and has the advantage of being very simple in composition; it is a class of rock termed an Oophytic diabase. From Rowley Regis and Cleve Hills we have another good stone, specially adapted to very heavy traffic on the outskirts of towns. It is a basic rock also, and usually contains a fair percentage of olivine, and often contains iron, manganese, and other minerals; in fact, we can usually expect where a stone has a low percentage of silica that there will be a marked increase of iron oxides, lime, soda, and magnesium, and in igneous rocks it is the silica which gives the compactness and hardness.

Sandstones are rarely sedimentary rocks; they vary much in hardness, some stones being so soft that they will crumple up by the least pressure. It is then hardly a stone at all. Sandstone is a composition of various hard substances such as quartz. These substances are bound together by a cementing matrix. This matrix is more often than not carbonate of lime and iron, and sometimes silica, which was introduced by percolating water; the size of the grain is a very important factor in the quality of the stone (*vide Sandstone* under Materials of Construction); the stones which contain the larger particles are termed *grit stones*. Sometimes they contain very large proportions; the stone is then known as *Breccia*. Sandstones of a hard quality are suitable for road metal. Sometimes muscovite is present in a large degree; under such conditions the stone will split longitudinally. It is then very suitable for paving flags; the best comes from Yorkshire; it is then, however, unfit for metal; the presence of felspar is an advantage. Sandstones are practically without exception disintegrated igneous rocks, and may therefore be expected (though such is not always the case) to contain many of the essential ingredients of igneous rocks, principally silica.

Sandstones which have only a small percentage of quartz have special names.

Greywacke is a variety; it is a cemented mass of quartz, felspar, and other minerals, while what is termed *Arkose* is practically disintegrated granite in a cemented form. Of course the cementing matrix is a very important element in sandstones when deciding upon them for road metal. Some sandstone is practically a compact mass of quartz; it is then called *Quartzite*, a form of stone much found in the Lickey Hills, near Bromsgrove in Worcestershire.

The cementing mass should be siliceous, but is very often not so, but of a calcareous nature; the stone will then lose its value as a road metal to a great extent; this is because calcium carbonate is soluble (*vide Lime*, Chapter XIII.). In some stones which have a calcareous matrix, this matrix will become crystallised, which enhances its value. *Kentish Rag* is a variety. When the matrix contains iron in the form of oxide we have ironstone, known by its brownish-red streaks.

Regarding the value of the various sandstones for metal, quartzite is the only really suitable variety. Most of the stones, however, make fairly good

flags; the common property of sandstones of hardening in the atmosphere after quarrying also adds to their value in the latter respect. In Ireland we have a good sandstone from County Clare, very similar to York stone, while from Kerry comes the Old Red Sandstone.

Limestones.—The principal constituent of limestones is calcium carbonate. It may reach as much as 98% of the composition; the origin is usually organic, viz. fossils, etc. (Chapter XIII.) The calcium carbonate having become crystallised by redispersion into the form of "calcite" or "Aragonite," the latter form is readily soluble. Dolomite is what is termed a double calcite; when present in a limestone it improves it as a road metal. *Aragonite*, however, is an undesirable quality; it weakens the stone.

The composition is usually fragments, together with a cementing matrix.

According as this matrix has silica or calcite in excess we have calcareous sandstones and siliceous limestones. This refers to calcareous grits. In all cases when deciding to use limestone as a road metal, we can say that the stability will depend upon how far the original aragonite has been replaced by crystalline calcite. Dolomitisation is the production of the double calcite, and is due to the presence of magnesia. Carboniferous limestone is a mixture of dolomite, calcite, and chalybite (or ferrous carbonate) is also present. An important limestone variety is *chert*. The portland stone is a good example. It may be generally stated that the presence of silica in limestone used as road metal is a distinct advantage.

The several varieties of limestone in common use are—

Silurian, from Coniston in the Lake district.

Devonian, from Devonshire.

Carboniferous, often called Mendip Granite.

Limestone is also plentiful in Ireland. It has the peculiar disadvantage, however, of being particularly prone to cavities (a natural source of weakness). This is due to the calcite having all weathered out, leaving the more durable dolomite. Oolitic limestones do not make good road metal; they are of value, however, in the preparation of kerbstones and paving setts.

There is a special variety of limestone from Scarborough. It is principally ironstone, viz. all the calcite has been replaced by chalybite. Lias limestone, which is used principally for making lime, which has marked hydraulic properties, is sometimes used for paving stones, especially that which comes from Somersetshire.

Flint occurs in nodules in the upper chalk formation, and is practically pure silica (about 98% by volume). It is considered by some to be very chemically stable, but has the disadvantage of being acted upon by alkalis. Flint, like sandstone, appears to improve on exposure. When first dug it has the appearance of being quite black; this, strange to say, is not the result of colouring matter, but due to the presence of what is called colloid silica, a mineral which has the property of absorbing the light. Flint is noticed to have a white powdery crust on the surface. It is composed of silica, and is due to the effects of the atmosphere upon the stone; its chemical composition is the same as that of the stone, however. Flint, when fresh, contains much water; this causes fresh-dug flint to be very brittle, but the weathering will greatly toughen the stone.

Chert is a massive compact form of amorphous silica. It is found in nodules amongst limestone rock. It is really an impure variety of flint, but it will make a better road stone than flint because of its greater toughness and less liability to pulverise. It is a rare stone, however, and hence its use

becomes expensive, Seven Oaks stone, in fact, being the only variety in practical use.

Regarding gravel when used on roads, Flint gravel is the most important variety. Others are, angular, sub-angular, and pebble gravel, and to those composed of fragments of igneous rock is given the name of *Glacial Gravels*.

When gravel is used on roads we want to carefully consider (1) the nature of the stone. If this is not of uniform hardness, it will prove very unsatisfactory, because the roadway will eventually wear to a very uneven surface.

(2) The size and shape of the stones. The angular variety is obviously the best. It must always be well screened before use.

(3) The nature of the finer material. This is usually sand, but is often clayey and ferruginous matter, and upon which the binding properties of the gravel will depend. Greywether sandstone is a compact gravel variety, because it does not occur in stratified masses. Shingle is coarse pebble gravel.

Artificial stones, more especially for flags and paving, are now coming into general use. Therefore it is quite necessary that a few words be said in reference to them. The material and cementing matrix, both of course, want very careful consideration.

Regarding the former, it must be free from magnetite and pyrites. Their presence would cause ugly stains. Crushed granite and syenite are the best, but we can also use with advantage hard gravel and freestone chips. Clinker from refuse destructors has also been adopted with much success.

To make artificial sandstone, clean sharp quartz should be used, and enough iron oxide added to give the colour.

Everything, however, depends on the cementing matrix. If this is bad the whole process will fail. We described fully the composition and essential qualities of Portland cement in Chapter XIII.

Victoria stone is widely used for paving flags, and generally will give good results. The flags should, if at all possible, be machine made. When made by hand, air bubbles cannot be avoided; but of course they can be greatly eliminated by good mixing and careful tamping with an iron rod in the mould. Trembling will also have a marked beneficial effect. An excellent wearing surface is usually produced. Another variety of Victoria stone is Stuart's **granolithic**. It is made from Leicestershire granite (vide Chapter XIII.).

There is also a special composition now on the market called "Granite Plumbic." The process of manufacture is the same as any ordinary artificial stone, but in addition to the ordinary mixture lead filings are included. These, by reason of their extra weight, will sink down to the bottom of the mould. The result is, if the stone when dry is laid bottom side up, we have a surface which has excellent wearing qualities, and has the advantage of providing a firm foothold for pedestrians in all weathers. Paving bricks are also used in some towns. They are not, however, in very general use. Being good conductors of heat, they are very tiring to pedestrians in hot weather. They must be subject to thorough vitrification (but in no case fusion), and they are very hard and wear well. They are, however, liable to crack. The best material for use in their manufacture is loamy clay. Stafford bricks are generally used (vide Chapter XIII.). (Vide note at end of Chapter).

Asphalt.—The use of asphalt on roads is now very general. It has the advantage of being—

1. Practically noiseless,
2. Very-sanitary,

3. Easily laid,
4. Moderately durable under ordinary traffic; and the disadvantages of being—

1. Slippery in wet weather,
2. Difficult to make a satisfactory repair after the road has once been "up."

The good and bad qualities of asphalt and its properties in general were treated in Chapter XIII.

When heavy traffic is to be provided for, it must be used in the compressed state, but for paths it may be laid in the mastic form by mixing about 70 per cent. of Trinidad bitumen with about 30 per cent. of grit.

Tar macadam which has lately been so much used is made with limestone or other soft rock. Granite and very hard rocks are unsuitable, as they work loose and are liable to form a bumpy surface. The best proportions for the matrix are as follows:—

Best gas-tar	34 gallons	} To which must be added 80 gallons of water.
Engine ashes	11 barrow loads	
Slaked lime (lias)	4 " "	
Grit	4-5 " "	

The dry materials are first mixed together and the water gradually added. When the resulting mass gets the consistency of mortar it must be passed through a pug mill and is then fit for use; the tar must be boiled for about 3 hours.

Wood Paving.—When wood is used for paving (and its use is very general in towns), the timber used should be of the Exogen variety (Chapter XIII.). The several advantages of wood paving are—

1. Noiselessness.
2. Good foothold in all weather.
3. May be used on the steepest gradients.
4. Fairly easy to repair when disturbed.

And disadvantages—

1. Cannot be said to be sanitary.
2. Wears to an uneven surface.

The most suitable timbers to use are—

Australian blue gum, Jarrah and Karri, because these woods do not require creosoting, and do not appreciably expand when wet. The softer kinds of wood will expand as much as $\frac{1}{4}$ in. in 9 ft. under the same conditions. Seasoning of such woods is not necessary, nor is it advisable. When hard woods are used a depth of $3\frac{1}{2}$ in. to 5 in. is quite sufficient (a saving over the softer woods in itself). The fibre must be quite vertical.

When softer woods are used for paving they must naturally have some form of preservation. This usually takes the form of creosoting (Chapter XIII.).

Regarding the quality of creosote used, "London" creosote is much favoured by engineers. It is made from Newcastle coal, and is very rich in heavy oils. "Country" creosote is made from Midland coal, and is a much lighter variety; whilst Scotch creosote, made from cannel coal, is lighter still. It is the duty of the engineer, when the work is let by contract, to see that no crude tar is added to the creosote. Neither should creosote have any traces of ammonia; its presence rots the fibre of the wood. On account of the large amount of naphthaline it contains, London creosote is used more than any other variety. It is easily detected, too, in a piece of wood, because it solidifies in the form of sparkling crystals.

When specifying creosote it must be stated that—

1. It must be entirely liquid at 100° Fahr.
2. No deposit must take place above 95°.
3. At 600°, 25 per cent. must still be undistilled.
4. 8 per cent. at least of tar acids must be present.

The other processes in vogue for preserving woods are, Bethell's, Boulton's, Payne's, and Kyanising. They are seldom used for paving blocks except Boulton's, in which case the creosote is heated up to 220° instead of 120° in ordinary creosoting before injection. The supposed advantage is that the extra heat will vaporise all the remaining moisture and aid complete vacuum. It is of great advantage when the timber to be treated is young.

The gradient of a road has a good deal of bearing upon what paving would be suitable. For instance, macadam could be used on a gradient as steep as 1 in 10, which is about the limit for ordinary traffic, and is seldom met with. Wood paving can be used on moderate slopes up to 1 in 26, while asphalt would be limited to roads whose gradient is less than 1 in 50. Sometimes conditions (such as economy for roads which have not very much traffic) will warrant the blending of two different kinds of stone.

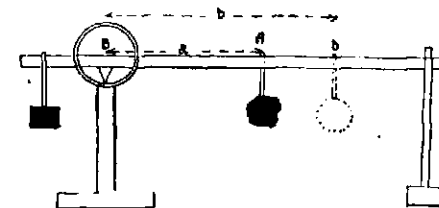


FIG. 391.

For instance, in an iron district limestone would be plentiful, but perhaps expensive, while blast-furnace slag would be easily obtained. The average composition of such slag would be about—silica 30 per cent., alumina 20 per cent., and lime 40 per cent., together with various impurities. Then if we are going to use slag and limestone together, they must be carefully laid in layers, with the limestone on top; this is to prevent an uneven surface forming. On the other hand, there is no doubt that the most satisfactory (to the ratepayers and engineer) results are obtained by the use of the hardest hand-broken stones; at the same time the cost will be at least twice that of the machine-broken stone.

The Testing of Road Material.—Testing samples of road metal is an important duty of the municipal engineer, but unfortunately is a practice which does not receive as much attention as it might (due, doubtless, to its lengthy and troublesome nature) at the hands of a person who, more than anyone (it may be said), has his hands full of numerous duties.

Of course important chemical tests on stones are made by experienced chemists. But a few simple tests can easily be carried out by the assistant in the office, and if properly done (which perhaps would not be the case if left to assistants, hence its conspicuous absence) would prove of great value.

To start with, there is the specific-gravity test. It is very useful, and is made on a simple machine similar to that shown in fig. 391. The sample being hung at A, so as to balance when dry, is immersed in water and placed at b. Then call the distance AB = a and Bb = b, we have a formula for specific gravity

$$= G = \frac{b}{b-a} \quad \dots \quad (241)$$

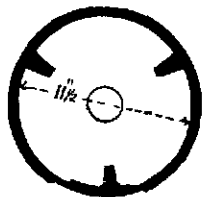
Specific gravity, however, is no true indication of a suitable road metal, but

knowledge of it is useful, in that it has been found from experiments that the crushing strength usually increases with the specific gravity. Again, we know 1000 oz. of water equals 1 cubic foot. Upon this assumption the following equation will give us the actual number of cubic ft. in a ton of that material, which is very useful to the engineer in making up his estimate when he knows the stone he is going to use.

The equation is

$$\frac{2240 \times 16}{1000 \times \text{sp. gr.}} \quad (242)$$

Owing, however, to the natural voids in a heap of stone, it is usual to take 55 per cent. of this as the correct quantity. Now in ordinary repairs to roads the usual quantity of road metal required per mile per annum = 100 cubic yards, therefore we see a heavier stone has less covering power per ton, and may come more expensive (stone being sold by the ton). On the other hand, a light stone may wear badly and become more expensive in the end. To discriminate between the two is the duty of the engineer, and it is in these little knotty problems that the engineer of experience has advantage over the younger members, and puts the university engineer (?) completely out of court.



Ribs Square
SECTIONAL AREA
FIG. 392.

The next test to consider is the absorption test. It likewise is very important.

The disadvantage of porous stones is that they will invariably fail and disintegrate under heavy frosts, especially flagstones. The test will directly determine the absorptive power, which has been found to vary from 1% in granite to 12% in some limestones. When using the balance shown by the diagram 391, we weigh the stone dry, then it is immersed for at least 24 hours in water and reweighed. Then having noted the particulars, the same as in reference to the specific-gravity test, we turn Equation 241 upside down so,

$$x = \frac{a - b}{b} \quad (243)$$

The result will be a decimal. This is the percentage of water in the specimen.

The resistance of stones to abrasion will depend on the tough quality of the stone rather than the hardness. It causes mud and dust. The action of acid (horse droppings, etc.) also aids the process. Unfortunately the abrasive test is not very reliable. The testing would be carried out in a cast-iron cylinder of section shown in fig 392. Four lbs. of stone is put in for each test, the duration of which would be about 7500 revolutions.

There are other ways of getting the same results; engineers usually adopt their own. Now these tests are to produce grit. The amount will, of course, vary with the material. The best basalt rock would only produce about 6-7%, while Portland cement might produce 87% when the test was carried on over a period of four hours. Flint and limestone will also produce chips. An additional test is to mix the dust so formed. After passing it through a sieve of 100 meshes to the square inch make this into a stiff paste. Then obtain a suitable press and form a block 1 in. in diameter x 1 in. thick. The pressure employed should be about 1150 lbs. per sq. in. Then allow a hammer

whose head weighs about 2 lbs. to fall $\frac{3}{4}$ of an inch on to the specimen till it breaks. The rebound of the hammer is recorded upon a diagram. It is when there is no rebound that the specimen is said to be broken. Quartzite and granite when so treated will be found to fail at about the third blow. Limestone, however, will be found to stand as many as 40 such blows without fracture. The reason for this, especially as it is the softer stone, is that limestone contains much lime and oxide of iron. These retard the breaking, forming a sort of natural cementing matrix. Another useful test, when a suitable apparatus to perform the previous one is not handy, is as follows. Obtain a 15-lb. hammer, get some specimens of the stone in question, which has been previously broken to pass through a 2½-in. ring. Allow the hammer to fall 10 in. to 12 in. on the specimen, and note how many such blows are required to break it into fragments which will pass through a sieve of ¼-in. mesh. Experiments show that this will, of course, vary. The limits would probably be reached in greenstone 902, and quartz 294. The crushing test is of practical value for building stones only.

Vitrified Brick.—A vitrified brick highway is the one kind of pavement for country roads that has passed the experimental stage, the most important special features that have been proved being the following,—they are the most economical, the most satisfactory in use, and their proper construction in all its details is easily understood.

It is true that they may, and generally do, cost about one-fourth more in the first instance than highways made of macadam and broken stone, even where they are treated by one of the various methods in the use of asphaltum, oil, or tar. The cost of a highway, however, is not to be measured by the first expenditure alone, but by the cost of the first expenditure plus the cost required to maintain it in first-class condition for the purposes for which it was built.

There is another element of cost that must be considered by way of comparison in the economy of a brick road with that of any form of gravel or macadam, and that is the deteriorated condition which always obtains for a considerable lapse of time before repairs can be effected. The writer recently witnessed a condition illustrating this element of cost. Six years ago, to the then corporate limits of the city of Linton, Ind., a brick street was constructed. Three years afterwards it was determined to construct a highway of broken stone from the terminal of this improvement some distance into the country. From the time the improvement was finished the highway was subjected to excessive traffic. For more than a year past a deteriorated condition of the stone portion of this road was such as to make it impossible to haul as much tonnage per vehicle by at least 50 per cent. as was hauled over the road when in good repair.

The author recalls another example of a macadam road constructed three years ago in northern Ohio. This road has proved so worthless in use that the farmers along its route have recently petitioned for a brick road.

The vitrified brick road is satisfactory in use because it is always in repair and practically free from dust. The brick road is the one road where legislation against its use is never necessary. In many States legislation of this sort against improved highways practically stops their use for weeks at a time.

A most important feature of vitrified brick roadways is that their proper construction in all details is easily understood. The essential ingredients that enter into all parts of the construction are not mystified in unknown chemical

as far as the writer's experience goes over a number of years, 10 to 12 ft. per hour, which includes ample time for changing the bits, cleaning the bore-hole, setting up and taking down the machine, etc. The rock on which this experience was gained was mainly whinstone of a fairly tough and even quality. Having drilled the hole, the next procedure is to thoroughly clean them out and wipe them dry, the latter being often impossible. They are then charged with the explosive. The use of gunpowder for this purpose has now gone out of vogue, and what are termed high-grade explosives have taken its place which are from 6-9 times as strong as gunpowder when considered as an explosive agent. They are generally of the nitroglycerine type, and have various names according to their special composition, such as gelignite, blasting gelatine, dynamite, etc.

Dynamite itself is a gelatinous compound. It is very rapid in action, and should be placed in the bore-hole in a plastic state. It must never freeze, but it is very liable to do so at temperatures as much as 10° above freezing-point. It is thawed by placing warm manure over it. The actual amount of the explosive used in each hole will of course depend on many things, such as the kind used, depth and distance apart of holes, nature of rock, etc. It is usual to have a plan and section of the bulk of rock to be dislodged; then from accurate measurement (having carefully marked each bore-hole in its true position on the plan), the amount of rock to be dislodged by each shot can be gauged with tolerable accuracy. Experience of the rock and the explosive used will then show how many tons of rock 1 lb. of that explosive is likely to dislodge, and the charge in each hole is calculated accordingly. The next process is placing the explosive in the hole, having wiped it as clean and dry as possible. The old system of ramming loose powder down the hole is now to a great extent obsolete, this charge being done up in specially made waterproof cartridges, which contain a known quantity. As many of these as required are placed in the hole, carefully on top of each other, and then the detonator. This is a thin copper tube, usually containing fulminate of mercury, and a flashing mixture, consisting of antimony sulphide and potassium chlorate, into which project the two bare ends of the electrical conductor. This is pushed into the primer, the hole being previously made therein to receive it. There is no doubt that if the detonator is placed in the centre of the charge a better blast will result, but it is generally considered that this advantage is outweighed by the difficulty in tamping the hole under such conditions. All holes, of course, will require tamping, and this is done either by wet clay, sand and (to a certain extent only in road stone quarries) water. Ramming must on no account proceed till a good depth of the sand or clay is inserted, and in any case great care must be taken in this to avoid injuring the insulation of the conductor.

When all the charges are properly set and tamped, they have to be electrically connected to the blasting machine, sometimes called an exploder. The machine essentially consists of a small dynamo, which is rotated at the required speed by hand, not, as may be supposed, by turning a handle like a telephone magneto, but by pulling up a Rack bar (hence the name of a certain machine). This dynamo is wound to give either a high or low voltage according to the purpose to which it is put.

In order to fire "tension" detonators, the high voltage is required. The voltage varies from 100 to 500. The general practice in quarry work is, however, not to use a high voltage, because there is no practical way of testing the connections on the high-tension principle before the blast, and hence there

is always risk of misfire. With the low-tension system, however, this is not so liable, when all the connections are tested immediately before the blast. There are two main systems of connecting up a series of shot-holes with the blasting machine, known as the "series" and "parallel" respectively, terms which will be fully appreciated by those who understand even the elements of electrical engineering. The series system is used in low-tension work, owing to the chances of misfire being reduced to a minimum, for reasons stated. The wires are usually not insulated, but must be of stout copper, according to the amount of the current. In high-tension work, however, the parallel system is invariably used, each shot-hole being thereby rendered entirely independent of the other. All insulation must be of the best, and the joints very soundly made. When all due precautions have been taken, and the lines laid and jointed carefully by an experienced man, and the best quality explosive detonators used, the risk of misfire should be reduced to a minimum. Unfortunately they do occur, and the removal of or attention to such a charge after attempt at firing is a dangerous procedure. The principal cause is defective detonators, especially in high-tension work, not because these detonators are of necessity more liable to defect, but from the inability to carry out preliminary tests by the operator. Of course, in electrical blasting, the disconnection of the blasting machine will materially lessen the danger of going near the shot-holes. With safety fuses, the great danger is what is technically termed *hanging fire*.

Assuming now that a satisfactory blast has taken place, the next thing to do is to prepare the stone for road metal. We have already pointed out the necessity of accurately determining the amount of explosive in each shot-hole to produce stone of a size just large enough to reduce the subsequent breaking to a minimum. No hard and fast rules can be laid down. Twelve tons of rock per lb. of dynamite is given as a fair average under ordinary conditions. If the operation is gauged to a nicety, the stone should be of such a size that it may be placed in the stonebreaker (machine breaking being usually employed in large quarries) with the least amount of sledging.

terms or secret processes, and the price can be readily obtained without difficulty.

Quarrying.—Having fully described the properties, nature, and composition of the road stones principally in use in Britain at the present day, we proceed to investigate the means whereby such stone is obtained, or "won," as it is technically termed. The smaller urban and rural councils usually obtain their supply of road metal by contract, but county councils, where at all possible, invariably own their own quarries. The selection of a quarry is a matter incapable of adequate treatment on paper; it must in all cases be left to an engineer of experience; suffice to say that there are three main considerations, as follows:—

1. The depth of superincumbent earth to be removed before the actual rock is reached, known as the "Tirr."
2. The opening up of the quarry so that the bed joints shall dip towards the quarry face, in order to allow of removal of the rock by gravitation.
3. The natural facilities for drainage without pumping, which is not always possible.

Hard rock for road purposes requires to be usually blasted, which in turn entails drilling. This may be done by hand or machine. Hand drilling, now rapidly going out of vogue, is done by means of "jumpers." These are bars of octagonal steel $\frac{1}{4}$ in. to $1\frac{1}{4}$ in. diameter, and in length up to 6 ft., according to the size of the hole, but a small length is the most effective. The process consists of striking the jumper with a sledge hammer, usually 4-5 lbs. in weight, but often as much as 10 lbs., the drill being turned slightly after each blow. The use of water in a bore-hole is essential to the proper drilling of the hole and the durability of the drill. The use of a "scraper" is resorted to to clear out the sludge from time to time. The actual amount of drilling done in a certain time will, of course, depend upon the quality of the stone, and the cost of the actual stone placed on the roads is, of course, greatly influenced by the former consideration. Mr. Thomas Aitken, in his *Road-making and Maintenance*, gives much valuable information on this subject, as the result of actual experience and records. He also recommends the computation to be made in cylindrical inches, a practice which has much to recommend its use in comparisons of labour, and the usual results in practice under the writer's supervision are as follows—

Whinstone	26 cylindrical inches.
Limestone	27 " "
Basalt	18 " "
Granite (Irish)	15 " "

which denotes the number of cylindrical inches drilled per hour. The cost will, of course, depend upon the wages paid.

Modern engineering practice, however, tends to machine drilling, a process of the same nature as hand drilling, that is, the striking of a number of blows in quick succession, and technically termed **percussive boring**.

Of makes of rock drills there are many, each of course claiming to have special advantages. Perhaps the best known are the Ingersoll, Sergeant, Hirnant, and Rio Tinto drills. A full and detailed description of each would be unnecessary here and take up too much space; suffice to say that the principle upon which they work is practically the same in each. The drill bit has, of course, to be subjected to two distinct motions like the hand drill that is, reciprocation and turning. The former is produced by the steam

compressed air in a similar way to the ordinary steam engine piston, except that the valve controlling the admission of the fluid is actuated by that fluid very much in the same way as a well-known direct-acting steam pump. It is essential, of course, to get the full blow of the drill-bit on the rock, that is, there must be no undue cushioning of the air or steam, and it is the capability of giving this full blow which is the special feature of the first-named make above. Again, the piston stroke of a drill must be capable of variation in length, so as to facilitate the starting of the holes. Again, it must not be supposed that the mere drawing the drill back on the back stroke is all that is necessary. In case of the drill sticking, which is usual enough in fissured rock, the steam or air should be capable of easily drawing it back.

The rotary motion is imparted on the back stroke by a rifle and free-wheel arrangement. The drill is mounted on a tripod, and the feeding forward as the hole progresses is done by a quick-threaded screw turned by the operator.

The tripods are of special construction to combine lightness and rigidity. They are kept in position by heavy weights.

The drill-bits used in hand boring are not suitable to machine work. Those used for the former purpose are chisel-pointed; they may, however, be used in machines, provided the stone is of a soft nature, such as sandstone, in which case the drill has two grooves which run upwards towards the shank along the centre; they facilitate the removal of the sludge. The usual forms used for this purpose are the cross drill and the X drill, the latter being the more efficient of the two, the resulting hole being generally truly cylindrical. The largest cross dimension of the drill should always be more than the diameter of the rod of which it is made, this size being maintained about 8 in. from the cutting edge.

To discuss here the best position for the resulting blast in which to place the bore holes is a matter incapable of adequate treatment, actual experience in quarrying operations being alone a safe guide, especially as the nature of each quarry requires special consideration. They are naturally sunk at the most efficient distance from the face of the rock to be removed, and the general effect of the blast is to dislodge a mass in the form of an equilateral triangle, having the shot-hole at the apex and the quarry face the base. This applies, however, to single shot-holes only. Vertical holes have usually the best effect when about 15 ft. deep. In any case, before a definite conclusion can be arrived at, the "tirr" has to be stripped. Then, besides the vertical holes, it is usually necessary to drill horizontal holes as well. They are termed "breast holes." The number of holes in a roadstone quarry are usually fairly numerous, because the more the rock is shattered the better, the subsequent sledging and breaking being thereby greatly reduced. Care must be taken, however, that the stone is not reduced too small. Although, in tunnelling operations and mining, the drills are worked by compressed air, the work in county and town council quarries is not usually enough to warrant the putting down of a compressed-air plant. Hence steam is the medium resorted to, and this, more often than not, is supplied by a steam roller which happens to be idle. Flexible tubing of the very best quality and swivel joints have to be employed. The great disadvantage is, of course, the wetness of the steam, but as it is rarely used expansively, this is not so serious as may be at first supposed. The use of a portable boiler or a road roller will, of course, lessen the evil, by bringing the steam as near as possible to the working face. The usual average practice as regards the rate of drilling is,

CHAPTER XIX.

THE CONSTRUCTION AND MAINTENANCE OF ROADS—continued.

Construction.—When making a new road, the site of such is, of course, the primary consideration. In Britain new roads of any length or importance are now rarely made. Long surveys and levels for a road are things of the past at home, new constructions being mainly short roads to open up residential districts on estates and on the outskirts of large towns. For the benefit of those who have, however, to lay out a new road as a line of communication, the following points are given.

1. The most direct road between two distant points is seldom that adopted in practice.
2. The gradients must be as easy as possible.
3. The construction must be economical, that is, bridges, cuttings, banks,

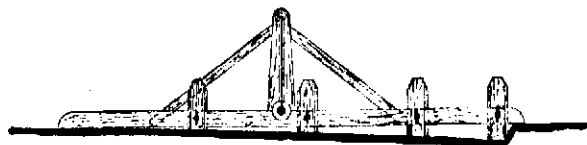


FIG. 393.

etc., all add to expense, and it must be remembered there is little or no return for the outlay on a road except general benefit to the district.

4. As many points of importance should be touched as possible without unduly lengthening the road. Then a survey is made and levels taken in the usual way, as described under the chapters in reference to the same. The gradient of a road is really limited by the angle at which an ordinary carriage once set in motion would descend with an increasing velocity. This on a paved road would limit the gradient to 1-45 (which is often exceeded), and on macadam 1-68 (still more often exceeded).

It is of great importance to make the cross section of the road very accurate. For this purpose a special form of level should be in the hands of every road overseer. It is shown in fig. 393. The small pegs or staves have slots in them. They are fastened through these slots by means of wing nuts, and can be raised or lowered at will to give the required curvature. The level must, of course, be set truly horizontal. That is what the plumb bob is provided for in the centre.

Ditches, when provided for drainage purposes, should be on the far side of

the fences from the road, and the cross drains, which should be numerous, must pass through the hedges. Such hedges must not be more than 5 ft. high and should be kept well clipped, so as not to rob the road of sunshine, which is most essential to its long life. A wet road is always an unsatisfactory piece of work and a source of continual expense. To attempt to repair a road which *never* had a solid foundation is absolutely hopeless, but in old and badly repaired roads we frequently find the foundation stones working their way to the surface. They must be promptly broken, on their first appearance, with a hammer and fresh metal put on.

Gradients were mentioned in Chapter II. in reference to roads. The effect of a gradient of 1-100 is practically nil on ordinary road traffic. A gradient of 1-10 would, however, take 4 times the energy to overcome it of a grade of 1-100. In flat country, grades above 1-30 should be avoided, but in hilly districts it may be necessary to go higher, but 1-20 should be the limit on main roads unless under exceptional engineering difficulties. Again, curves on the flat should be limited to 50 ft. in radius, but in hilly districts this may have to be exceeded to as small as 20 ft. radius. The underlying strata is now the first thing to consider when the main route has been decided upon, and levels taken and sections plotted in accordance with the principles previously set forth.

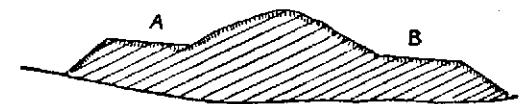


FIG. 394.

All marshy, boggy land, and any land liable to floods, unless the road is upon embankment, should be avoided. When a road is laid over a change in the strata, that is, where impervious and pervious strata meet, the road will always be liable to be wet (*vide* Land Drainage). Regarding roads upon a hillside, the nature of the strata here will be a very important consideration. Take fig. 394. The diagonal lines show the direction of the planes in the sub-lying strata. The road if made at A would be always liable to slips in wet weather, and would necessitate heavy engineering expenditure, while that at B would be quite safe. Such a consideration might easily alter the route of a road. Of course, slipping at A would only occur when the inclination of the sub-strata was at an angle greater than the natural angle of repose of the particular material.

When deciding where to place bridges across rivers, etc., and railways, we must bear in mind that—

1. The bridge should not be approached on a stiff grade.
2. The bridge should not be approached on a stiff curve.
3. Skew bridges are well avoided if possible. They are expensive to build.

Regarding the resistance to traction upon roads, it has been found by experiment that the average resistance set up by a good macadam road to traction would be $\frac{1}{10}$ of the gross load = f . Other values of f are found in comparison to be—

Stone pavements (setts)	$\frac{1}{27}$
Broken-stone road (firm foundation)	$\frac{1}{15}$
" " (unstable foundation)	$\frac{1}{32}$
Gravel road	$\frac{1}{18}$
Soft gravel and sandy roads	$\frac{1}{7}$

Regarding the formation of roads in general, works of earth and masonry seldom present themselves in the same way as they do on railways, for the simple reason that much stiffer gradients are allowed, and such works are avoided on account of expense.

The formation, in fact, usually consists of digging a drain on each side of the road if the ground has a level cross section, and one on the uphill side if it slopes. The earth so dug should be placed upon the proposed road surface when the sods have been removed, to raise that surface so as to provide ample sunshine. Or it may be placed in banks between the ditches and the road, and these banks planted with hedges, and the sods previously removed carefully placed thereon and grass seed sown.

On wet ground the foundation should be that of **Fascines**. The process is as follows. The road bed is excavated to about 3 ft. deep. Upon this is laid a filling of clean sand or gravel. Upon this again are placed the fascines, which are usually bundles of twigs about 20 ft. long and 9 in. to 12 in. in diameter. They are laid alternately lengthwise and crosswise till the bed is about 18 ft. deep. They are fastened by stout pegs. Such a procedure is, however, only

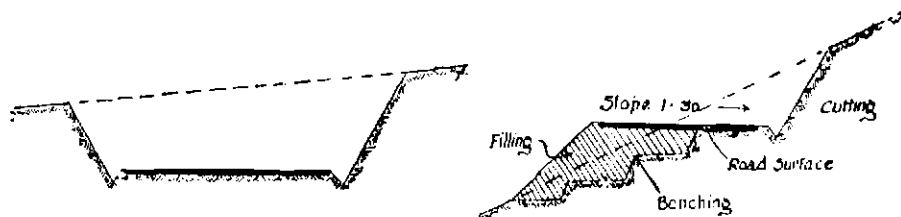


FIG. 395.

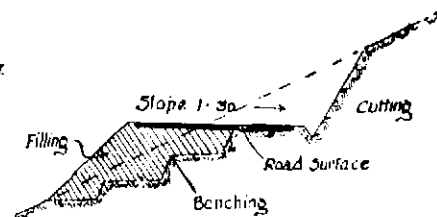


FIG. 396.

efficient when the fascines are constantly kept wet. Gravel is then spread over the top in layers.

When considering the distribution of gradients apart from the equalising of cutting and bank, it should be aimed as far as practicable to start long inclines stiffly and to provide frequent level reaches. This will put horse traction to the best advantage, the cutting and bank will be distributed precisely in the same way as stated in the chapter on Earthworks, carefully bearing in mind that clay when put into bank will shrink $\frac{1}{10}$ and gravel $\frac{1}{12}$, while large rock will expand $\frac{1}{12}$ and small rock $\frac{3}{8}$.

Sections of roads are usually drawn to scale of 5 chains = 1 in. horizontally, and 20 ft. = 1 in. vertically. The centre line must be carefully pegged out, and the edges of the road *lock-spitted* over the entire length. When roads occur in cuttings of any depth, the road must have plenty of sunshine, as before advised. The proper way to form such a road is as in fig. 395. The process is called **benching**. Roads when formed on a hillside are shown in fig. 396. If, however, the road has a stiff gradient, a better form is that shown by fig. 397, using retaining-walls where required. When making banks for roads during actual construction, they should be kept in the form shown by fig. 398. Then when the work has been completed they would be trimmed off as shown by the straight lines. In all cases the banks should be allowed to settle to the full height before any excavation for the road bed is begun. Again, hillside roads must always have a slope inwards of about 1-30. This is to allow for drainage, and to prevent the formation

of gullies on the outer slopes. When the formation is dug to the required form a 10-12 ton roller is then passed over it to make it quite solid, weak places being carefully filled and rammed. A suitable camber must be provided, as will be described when treating the road surface. Less camber is required, however, on roads which have a fairly stiff gradient. Regarding under-drainage this should not usually be required. In sandy soil it is quite unnecessary, but on clay soils it is required. It is, however, an easy matter (*vide Land Drainage*). Silt, however, will present some difficulty. The reason is this: silt, while being very retentive of moisture, is also very absorptive. A wet foundation is, of course, highly detrimental to a road. It causes failure by disintegration, chemical action, and, worse than all, frost, which would successfully break up a whole stretch of roadway.

It is usually when a road happens to cross the natural drainage system

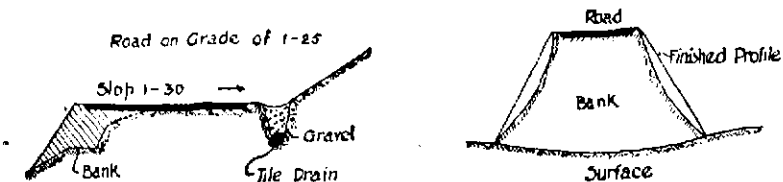


FIG. 397.

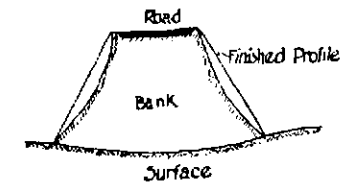


FIG. 398.

that such sub-drainage is required. Water will rarely percolate a good macadam road.

Regarding the drains used for roads, we shall treat the subject under Land Drainage. The usual forms are the blind drain, tile drain, square box drain, triangular box drain, and the shoulder drain, used in stiff clay. On slopes, cross-mitred drains are used; the apex of the V must point up the hill and the angle is generally large. A fall of 1-100 is generally considered sufficient for road drains. When springs appear in cuttings, which they frequently do, an outlet has to be provided for them. It takes the form of brushwood laid in a trench. These are covered with sods laid face downwards. The trench should then be filled in and well rammed. Blind drains will also answer the same purpose. Stone drains, of the form shown in fig. 399 and fully described under Earthworks, are often used. They are greatly used in railway cuttings. They must, however, be carried well below the formation level, so as to have a good foundation. All drains are liable to chokage by tree roots if they are of the types enumerated above, and this must be guarded against where possible by cutting off such roots. Those which will give the most trouble are the willow, ash, poplar, and elder trees, which have a great avidity for moisture. Where it is impossible to avoid them, it is best to provide fireclay sewer pipes jointed with cement. Culverts may be of stoneware, fireclay, or cast iron. If very large, brick or stone culverts may have to be provided (or reinforced concrete could be used). Their size will, of course, want careful consideration, depending principally on the rainfall of the district.

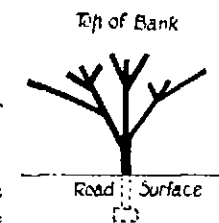


FIG. 399.

The Formation of Roads.

Gravel Roads.—They are first excavated to the required curvature and rolled down to a firm bed. Then 6 in. of large gravel should be put on and rolled in. Gravel for the purpose should contain a small percentage of loam. It must be carefully screened to three qualities, 2½ in. gauge, 2-in. gauge, and fine stuff. These are then put on in layers as follows:—

4 in. of 2½-in. gravel, 3 in. of 2-in. gravel, and 2 in. of fine stuff. We may, however, form a gravel road on a previously hand-pitched foundation. The old practice of road surveyors was to provide no solid foundation, on the assumption that a certain amount of "spring" in the road coating itself was an advantage. Such was the theory of James Loudon Macadam, the great pioneer of roads. His theory has, however, been found by present-day engineers not to have the several advantages its author held it to have, and the general practice is to form a solid foundation before any metalling is put on whatsoever. The best way to do this, and the way generally adopted, is to form a hand-pitched pavement similar to that proposed and adopted by Telford, whose roads met with such success. First of all, we excavate the road bed to the required curvature, or, better still, quite flat. It may with advantage be rolled, but such is not absolutely necessary. The hand-pitched

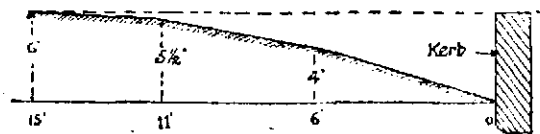


FIG. 400.

paving stones, which should be 8–12 in. long, 4 in. wide, and 5–8 in. deep, are laid thereon. It being the usual practice to lay the stones on a flat bed, we have to put the deepest 8-in. stones in the centre and gradually diminish the depth towards the sides. The stones should in all cases have a broad, flat bed, and be carefully hand-laid on this bed and rammed firm; when the pitching is done the men go over all the stones with a hammer and knock off the projecting pieces, with which the interstices are filled, and the whole made in a good condition to take the metalling and of the proper curvature. The engineer should then very carefully go over the whole to see that it is in good order. If that is so, the metalling may be put on. When the road bed is wet, the process of laying fascines or brushwood has to be resorted to; if, however, it is not very wet, at the same time not exactly what we might call dry, and sand is plentiful, a layer spread thereon before the pitching is done will prove a distinct advantage. This bed in ordinary practice is excavated to 17 in. below the proposed finished surface, and the hand-pitched pavement would be increased to 9–10 in. On top of this is placed 7 in. of fairly large 2½-in. broken metal, this is rolled carefully in two layers of 2½ in. each and one layer of 2 in. (steam rolling will be treated separately), then upon this comes a layer of sand and gravel about ½ in. thick. During the latter stages of the rolling, the road overseer has to see that the required curvature is being formed; this he does by means of the level before referred to (fig. 393). The form of surface which has been found to act best in practice is that of a flat ellipse, and this should be strictly adhered to. It is set out as in fig. 400, which is purposely drawn to a distorted scale; the top of the road at the

centre must be level with the top of the kerbstones, and these must show 6 in. on the face. Starting with 6 in. below the top of the curve at the channel in a 30-foot roadway, at a distance of 6 ft. we would have a rise of 4 in., at 11 ft. 5½ in., and finally at 15 ft. the maximum rise of 6 in. All roads when laid must be provided with properly formed channels to carry off the surface water. They must be very carefully laid to the proper gradient; they are made in various forms, setts, large paving stones, shingle stones, and specially formed stoneware blocks all having their several advantages. In any case a firm foundation should be provided. The best jointing material is made of gravel and creosote oil; it is preferable to cement. Coal-tar and pitch are often used. These

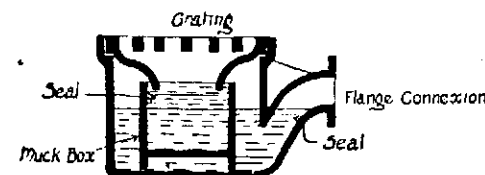


FIG. 401.

channels in towns where there is a main sewerage system will deliver into gullies connected with the main sewers; their forms are numerous, but simplicity of construction, resistance to chokage, ease of cleansing, durability and cost, all go to recommend the cast-iron gully. A suitable form is illustrated in fig. 401 in section. It will be seen that there is a double seal; they must be carefully set on a firm foundation of concrete. In country districts concrete cesspits are provided; they may also be formed of masonry or brickwork, and are necessary usually at the foot of steep inclines, where there is a large volume of water to contend with in the case of floods. A suitable form is shown in fig. 402. It can accommodate a larger amount of

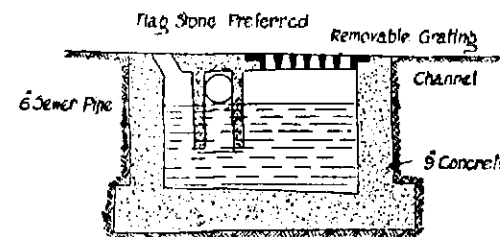


FIG. 402.

detritus. It is of course the duty of the surveyor to see that such gullies are periodically cleansed so as to be always capable of taking storm water, and they will require filling with water during very dry weather to prevent a nuisance due to bad smells. In towns where any pretence is made of a decent footpath, kerbstones have to be provided; they are usually of stone.* The best size and form is that shown in fig. 404. Unlike channel stones, which must be set on either a bed of concrete, sand, or gravel, they may be laid on the natural surface if quite firm, but a concrete bed is an advantage; the joints would be made with cement mortar in exactly the same way as masonry walls, and afterwards pointed (*vide* Masonry and Brickwork).

Considering now the much-favoured tar-macadam, its great points of value are its freedom from noise and impermeability to water. It is an excellent medium for footways. The stones should be of limestone, and not too hard (as before stated). It is best to mix it with a few blast-furnace clinders. The actual stone must be treated before use. It would be laid in layers. The first one would be of 2½-in. stone and heated together with either (1) pitch and coal-tar, in the proportion of 3 of pitch to 1 of coal-tar, or (2) 7 of pitch to

* Machine-made concrete blocks are now much used. They are cheap and easily made, but are not suitable for severe winter climates.

1 of creosote oil. Then layer number 2 would be applied when the first has been well rolled in, and would be of 2-in. stone mixed in the same way. Finally, a layer of small stones $\frac{1}{2}$ in. gauge is applied, and sharp grit added, and the whole finally rolled down to a hard surface.

Some road surveyors, however, consider it best to lay tar macadam as follows. The macadam is laid dry and just lightly rolled, then a mixture of 4 parts of mineral pitch to 1 of creosote oil is added, and sand and small chippings brushed into it while pouring. The author does not, however, consider it quite so satisfactory as the former method. One disadvantage of

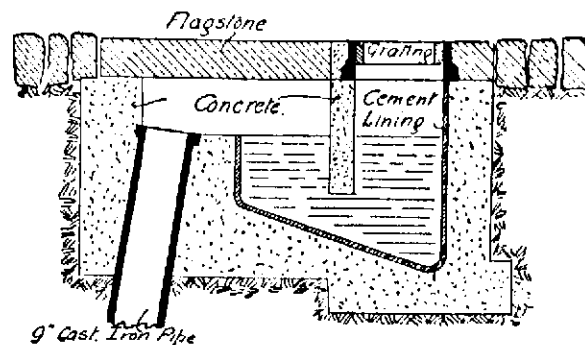


FIG. 403.—Built Gully Pit.

tar-macadam is that it must always be laid in *dry* weather. Its life will vary according to traffic from 3 to 10 years.

Sett Paving.—This form of roadway paving still continues to hold its own in large towns for heavy traffic; its wear, durability, ease of traction, and firm foothold for horses are advantages not easily improved upon. It is also impervious when well laid, easy to clean, and easy to repair. The one thing essential when starting to construct a sett pavement is a *solid foundation*.

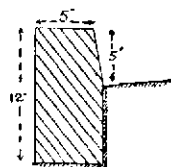


FIG. 404.

Gravel has been tried, but it must be very thick and well rolled. It is not, however, very efficient for the purpose. Some have tried the Telford hand-pitched pavement; in some cases it may be said to have given satisfactory results; again, tar-macadam has given very good results; but, on the whole, there is nothing to beat a good concrete bed for the purpose, and such is now the general practice. It should be 6 in. deep at the least, and should consist of 40% broken stones, 53% gravel, and 7% of best Portland

cement, and would cost about 3s. 9d. per square yard; then the actual setts would probably be $7\frac{1}{2} \times 7\frac{1}{2} \times 3$. They would then be laid across the street lengthwise, for obvious reasons. This, strange to say, however, has been found to hasten the wear of the setts. They are laid, starting at the edges, towards the centre; each row is then tightened up by means of a crown stone. On top of the bed and immediately below the setts must be a layer of sand to act as a cushion. When *bedded* rock is used the setts are cut along the line of the laminations; this practically limits the size to 9×4 . Hard granite is not used because it wears to a very slippery surface. The most recent practice in sett paving is that of using cubes about 3"-5" in size. The practice appears to have met with great success; they certainly tend to

a much smoother surface and eliminate a large amount of the noise (the principal disadvantage of setts); they are, however, expensive, both to lay and to lay, in consequence of their smaller superficial area. All courses of setts, whether oblong or cubes, must be laid like brickwork to break joint properly, though no special bond is required. They are rammed carefully into place and the road surface finished off to the proper curvature (which of course is obtained by the proper finished state of the concrete bed). They are then grouted up with coal-tar cement, which is nearly always used in preference to Portland cement. Its proportions should be

either 1 of pitch to 1 of tar (a)
or 3 of pitch to 1 of creosote oil (b) }

Before pouring in, however, fine gravel or chippings (which must be very clean) are brushed over the stones and into the joints. Sometimes a preliminary wash of cement grout is also used with them; the tar is then very carefully poured into each joint by hand separately.

Wood Pavements.—For wood paving, the same as setts, it is most essential to have a hard, unyielding, impervious bed. In all cases it should be of concrete, finished to a hard, smooth surface; the aggregate should consist of 2-1 pebbles and sand (2 of pebbles and 1 of sand, of course); the mixture should not be weaker than 7-1. On top of this we can either provide an ordinary sand cushion or a layer of mastic asphalt; in any case a layer of sand should be provided, but the sand cushion alone is especially efficient with pitch grouting. Asphalt may also be used. The blocks must be packed as closely together as possible. All open joints must be avoided; we also have to provide for an expansion joint. This is usually a layer of asphalt abutting on each kerbstone.

Open joints are still adhered to by some surveyors, but the author cannot himself see any advantages in their use. When decided upon, however, the best way to fill them is to pour in bitumen till the joint is nearly one-half full, and then grout up with cement and sand (1-1 mixture). Pitch grouting is, however, more advisable, for many reasons. They are—

1. It can be used with close joints.
2. It will materially help the expansion.
3. The blocks have decidedly less tendency to work loose.
4. It sets practically instantaneously.

Asphalt Pavements (*vide* Chapter XIII.).—As before stated, this is a form of surface now very much in favour in large towns where the gradients are necessarily small. It is also of much value for footpaths, and finds many other uses too at the engineer's hands, such as flooring basements, water-proofing underground structures, and the lining of tanks and reservoirs.

Like wood and sett paving, it must be laid on a firm foundation; 6 in. of concrete is the best. The essential qualities of the actual asphalt have been touched upon before; in addition, it is essential to pass all rock asphalt before use through a sieve of $\frac{1}{16}$ -in. mesh. The use of reboiled old asphalt is false economy. In laying, it is first of all heated, and then brought on to the site and laid, and rammed with a 10-lb. rammer. The rammer should also be heated, and the ramming very lightly done at first. It is then smoothed up by a special implement, and finally rolled with a half-ton roller. It has been found by experiment that, owing to the natural resilience of asphalt when new, practically no surface wear will take place when first laid for about two years. During that time it is continually solidifying.

destroyed—ground into dust or mud, and removed as refuse by the sweepers; and when, with their angles rounded off, the stones are worn into the worst possible shape for consolidation, they are stamped into place to form a real surface which no mending, filling up of hollows, or other expensive attention will make smooth and durable, and over which the first dry weather of summer will again set the loose stones rolling.

The next point for consideration is the binding material. A road which is not intended to be steam-rolled does, of course, not require any binding material; the wear and tear produces quite enough at the expense of the road metal. On the other hand, it is quite possible that the stone may be of such a tough quality as *not* to provide it. It is a great mistake to suppose binding is required in the body of the road. It is only necessary on the surface towards the final solidification by rolling. It should only be laid in layers of about $\frac{1}{2}$ of an inch thick. The best material to use is screenings of the actual road metal, to which is added a little sand; the prevalent practice in urban districts is to use road scrapings. There is no distinct disadvantage in using them, provided they are well weathered and quite free from objectionable foul matter. Chalk may be used very sparingly, but never with limestone, which should have sand only. Clay or loam must never be used. Superficial binding will, of course, render a road less permeable, hence it will be less liable to disintegration by frost. When, however, pebbles and shingle are the only road materials at the surveyor's hand, binding has to be copiously used. The two main actions of a steam roller may be taken as—

1. Crowding of the stones together.
2. Crushing the stones.

Obviously we want as much of the former as we can get and as little of the latter. The quality of the stone will settle the weight question.

For limestone and Kentish rag a 10-ton roller would be quite enough. On the other hand, hard, tough, igneous rocks would want quite 14 tons to consolidate them. But the use of a heavy roller requires consideration, for the road bed must be absolutely *unyielding*, and the surveyor must also make quite sure the heavy roller will not damage his sewers, water mains, or gas mains.

The Cost of Steam Road-Rolling.—The cost of steam road-rolling and the amount of work done in a given time vary with the systems of working and the principle of calculating chargeable expenses. Some surveyors are content to consider the daily cost as made up of wages, coal, and oil for the roller alone; others add a percentage on the cost of the machine to cover depreciation and interest; others, again, include the collateral expenses of watching, watering, and sweeping in the sand, etc., sometimes even adding the cost of the hoggin to the account. The statements of work done in a given time show equal disparity, as so much depends upon the thickness of the metal to be rolled, the proportion of sand or hoggin rolled in with it, the quantity of water used in rolling, the gradients of the roads, the degree of consolidation required for the traffic, and the number and gravity of the interruptions met with.

If the arrangements, however, are such as to secure the continuous working of the roller, by having stretches of road always prepared in advance of it, the number of square yards rolled in one day of 10 hours may be put down as between 1000 and 2000, according to the weight of the machine and to the influence of the other conditions on the work; and if the items constituting the cost of rolling be restricted to the wages of the driver, the bills for

coke and oil, and a sum of 10s. to 15s. to cover interest, wear and tear, the cost of rolling the above number of yards will average 22s. to 25s. In other words, from 3 to 8 superficial yards can be rolled and thoroughly consolidated for one penny.

Perhaps the best way to go about comparing the cost of steam rolling is to consider the experience and results of the engineers. A few abstracts appear as follows:—

The Surveyor to the Vestry of St George, Hanover Square.

An area of 1160 superficial yards of macadam road, newly lifted and coated, can in a day of 10 hours be sufficiently consolidated to admit of heavy traffic passing over it without disturbing the road surface.

The Borough Surveyor, Brighton.

Cost of fuel, water, wages, repairs, wear and tear, interest, etc., for the steam roller is £3, 1s. 1½d. for each day of ten hours actually at work on roads, or 6s. 1½d. per hour. About 200 square yards of road covered with the average thickness of flints are rolled in an hour, showing that each yard of flints rolled cost 0·366d. for the work done by the roller.

The City Surveyor, Edinburgh.

Up to the end of July I had laid down and rolled 10,000 tons of metal, covering an area of 100,000 superficial yards of roadway, at a cost (including all expenses) of ½d. per yard super. I have rolled 2500 superficial yards of 3 in. or 4-in. metal in a day, but the mean surface rolled may be stated as 1000 yards for town roads and 2000 yards for country roads, where there is no interruption to the working of the roller.

The Borough Engineer and Surveyor, Lytham (1903).

The roller has been in use for the past sixteen years, and during the past five years hardly been idle for a day, except for repairs, as, besides the rolling, scarifying, and breaking stone for the public roads, we have also rolled over thirty new streets during the past five years. Three years ago the roller was generally overhauled and the wheels and fire-box renewed, but apart from this the cost for repairs has been very small, and the roller has given every satisfaction.

	s.	d.
Engine-driver	5	0
Assistant	3	4
Fuel	1	6
Oil, etc., 8d.; Sundries, 4d.	1	0

Cost per day 10 10

Average cost of repairs per annum, including renewal of wheels and new fire-box, etc., three years ago,—£20.

Average weight of metal consolidated per annum, 3500 tons, besides 2000 tons for rubble foundations.

Average number of days worked, 200 rolling, 30 scarifying.

Road Rolling.—Nowadays no road can be considered complete unless it is steam-rolled. Most districts have their own rollers. Those which have not usually let the work by contract or hire a roller. Even if the surveyor has not had a mechanical training, which he will be at a great disadvantage without, the knowledge of the essential parts of his roller is quite necessary. By the courtesy of Messrs Aveling & Porter of Rochester, who have supplied rollers to practically the whole world, I am, by reason of the information which they kindly laid at my disposal, able to reproduce the following illustration, figs. 405 and 406, of a compound roller of 12½ tons. The advantages of a compound engine will be understood by the mechanically trained engineer. They are silent, steady, and economical of fuel.

This is an adaptation of their ordinary road locomotive for the purpose of road rolling chiefly, but intended also for picking up or lifting old roads, driving a stonebreaking machine, and drawing the stones to and from the roadside or other depôts. The principle adopted in the construction reduces, as far as is practical and consistent with efficient working, the bearing surface of the roller on the road, in order to give the greatest possible amount of compressive force of which its aggregate weight is capable. While the general arrangement of a road locomotive is adhered to, the driving wheels are widened to form the side rollers, the intervening space being covered by another pair of rollers carrying the front of the engine, which serve as steering wheels. These latter rollers are so connected to the engine that they accommodate themselves to the curved surface of the road.

The cylinder of the engine, which may be either single or compound, is placed on the forward part of the boiler, and surrounded by a steam jacket in direct communication with it, the steam being taken from a dome surmounting the jacket. Priming is by this means prevented, steam pipes are avoided, and important economy in fuel is ensured. The crank-shaft brackets are formed out of the side-plates of the fire-box, which extend upwards and backwards in one piece, so as not only to carry the crank-shaft, but to provide bearings also for the counter-shaft and driving axle in the most convenient position. This arrangement produces a combination of much strength, reduces to a minimum the loss and annoyance from leakage at strained bolt-holes, and unites all parts peculiarly exposed to injury by jarring, with so much firmness as to give almost absolute security against such injury. The crank and intermediate gearing is arranged to work between (instead of outside) the steel-plate brackets, and the fly-wheel is fixed close to the crank-shaft bearings for driving fixed machinery. This recent great improvement in the design of the roller not only adds to the strength of its working parts, but, by decreasing the rolling width, increases its compressing power. All the gearing is of the best cast steel. The roller is fitted with an injector as well as a pump, and a friction brake is worked by a hand-wheel and screw from the footplate, giving the driver powerful and instant control over the machine.

The boiler is constructed of best steel plates, flanged and riveted by hydraulic power; it is of the usual horizontal locomotive type, fitted with tubes, lagged with best hair felt, wood, and sheet-iron, and tested by hydraulic pressure. The fire-box is of special "fire-box" brand steel, and strongly stayed; the tender, of steel, is fitted with water-tank and coal-bunker.

The crank-shaft, counter-shafts, and driving axle are of steel. The four rollers are made of hard and tough iron, and the two driving rollers have a series of holes at convenient distances in their tyres for the insertion of picking-up spikes, when wanted. These driving rollers can be made with

renewable tyres or rims if required. The plan is substantially the same as that designed many years ago by Aveling & Porter, Limited, for a continental demand.

The larger rollers are geared for two speeds. They can work forward or backward with equal facility, can be turned completely round in a circle the diameter of which only slightly exceeds the machine's own length, and can roll hills as steep as 1 in 7 without detriment to the fire-box. As the steering is effected from the driver's footplate, all the management of the engine is in the hands of one man.

Road Scarifiers.—It is customary in repairing roads to loosen the superficial layer of old metalling by hand-picking before spreading on new stones. This is done as well to utilise some of the good material in the old road as to obtain a readily compressible bed for the new material. It aids also in levelling and giving form to the surface. Many roads have such stores of good metalling at or immediately beneath the surface that to repair them it suffices to pick up or lift the upper stratum from time to time, level, and roll it down again. When new material is added, a layer one stone in thickness is all that is requisite.

Spikes inserted at intervals in one of the driving wheels of a roller used to do this picking up rapidly and economically as compared with hand-picking, but more than one contrivance has lately been hit upon for still further cheapening the process. The simplest, strongest, and in every respect best of these contrivances is the Scarifier invented and patented by Mr Morrison, and manufactured by Aveling & Porter, Limited.

This machine (see figs. 405 and 406) is adapted for fixing to any self-moving engine, but especially to a steam road-roller; for, being permanently attached, it can be used at a moment's notice as economically for the slightest patching as for continuous work.

It can be driven close to the channels of the road without the roller wheels touching them; it will work with equal facility and advantage backwards and forwards; and, requiring neither belting, shafting, nor complicated gearing, it will not readily get out of order.

A 10-ton roller will work it with ease, and break up from 4000 to 5000 sq. yds. *per diem*.

See also Mr Evans' remarks on p. 277.

Every year the custom of rolling new and repaired roads before throwing them open to the public becomes more general.

It is easy to note the difference in appearance between the road that has been made by the traffic and that which has been finished by the roller, or to feel it when driving, cycling, or walking over, either; it is easy also to understand that the superior hardness and smoothness of the rolled road must tend, through lessened friction, not alone to the comfort of the public, but to economy in road metal. For let it be considered or remembered that the interspaces in a layer of newly broken stones occupy, when the stones are first loosely spread upon the road, about one-half the area covered by them, but only one-fourth the area after they have been settled into the compactness of an ordinary road surface. This compactness has to be attained by compression or by wear, by a heavy roller or by the traffic. In the former case the stones are driven into a compressible bed, with their sharp ends downwards and their flat sides uppermost, and form a level, regular, and solid pavement of interlocked angular stones; in the latter case they are rolled and kicked about by the traffic until at least one-third of the metalling is

road in a moist state. But mud must always be removed in quite a wet state. When this is attempted after it has become dry, the road stones will be loosened.

The causes of mud are not always so easily determined as may be supposed. It may work up from the subsoil on a bad road. Efficient surface drainage is, of course, a very important factor. Roads over bogs are always more or less liable to complete collapse. Their construction is expensive. Longitudinal drains must be cut on each side of the road 10 ft. back from its edges, with frequent cross drains. The foundation of sods and fascines has been described. In towns much damage will occur on roads where gas or water openings, etc., have not been properly and immediately repaired. It is the duty of every surveyor to see that this is very carefully attended to. His own men should do it, the labour and material being charged for if necessary. The filling must be replaced in the original order in which it was excavated, well rammed and no rubbish added.

As stated before, very little repairing will be required on asphalt roads or pavement for the first two years or so. It is usual, however, to add a

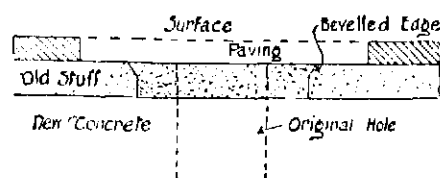


FIG. 408.

clause in the specification, when the work is let by contract, that the contractor shall undertake all repairs for a certain period at his own expense, and then another period at a fixed rate. This will ensure the best workmanship. The only real damage which should happen it in its early days, like that of all paved streets, is the breaking up for openings connected with water and gas supplies, etc. The practice of laying these mains all in a subway is now much in favour of municipal engineers; its expense, however, would confine it to the largest cities. When making such openings the repairs want great care. The way to make a satisfactory job of them is illustrated in the diagram, fig. 408, all broken concrete being carefully cut away all round, and four days at least allowed for setting.

We have already considered the cost of steam rolling as applied to roads. It remains, however, to see how we can arrive at a reasonable estimate of a new road constructed *ab initio*. A detailed estimate is, of course, the business of the surveyor again. He will, by his experience, be able to take local circumstances into consideration, and the general cost of such road will vary very much according to natural features, cost of material and labour, and transit. We can only here give a few general cases founded upon actual experience in practice.

Paved Roads.—Granite setts would cost about 14s. per square yard as against 3s. 3d. for macadam. This includes the concrete bed. On the other hand the cost of annual maintenance would be from 75%–80% less than macadam, and if the traffic is very heavy a greater saving still will result. Good setts can be usually obtained on an average of £1 per ton. Old setts can be cleaned by hand at 10½d. per sq. yd., by boiling at 4d. per sq. yd., and we can credit 2s. per sq. yd. by the use of the actual setts.

Hard-wood paving can be laid for about 11s. 6d. But if we include a foundation, 14s. will be a likely figure per sq. yd.

If soft wood is used creosoted, 8s. would be a reasonable figure; on the

other hand, their life is only half that of the hard-wood setts, upon which no wear need be anticipated for the first 3 years. The maintenance would work out at about 4½d.–5d. per sq. yd. per annum. The actual blocks cost about £9 to £10 per 1000, of which 5s. would be taken up in sawing them by motive-power.

Asphalt can be laid at about 12s. 6d. per sq. yd., including a good foundation.

The annual cost of repairing roads in urban districts has been found by comparison to be about £140 per mile per annum. Those in rural districts would cost £58 likewise, and lanes and byways £30. In Ireland an average of the reports of county surveyors sets the maintenance figure as low as £15, 10s.

Again, 1s. 7d. per sq. yd. per annum is a good average. Another estimate is 11½d., made up as follows:—

Stocking		d.	
Stone		8½	
Sand		1½	
Water		¼	
Rolling		½	
			} per sq. yd., which
			} would include
			} horse labour.

Tar-macadam could be satisfactorily laid for 9s. 3d. per cub. yd., or 1s. 6½d. per sq. yd., 4 in. deep. The following are the proportions of 50 cub. yds. of stuff:—

53 cub. yds. of pit gravel		at 3s. 6d.
82 gallons of tar		„ 2½d.
240 lbs. of pitch		„ 4 lb. = 1 penny.
89 bushels of coke		„ 3½d.
35 „ breeze		„ 3½d.
Wages on mixing the same		£7, 12s.
Actual laying of 1 supl. yd.		3s.
Total labour and material		4s. 6d. per sq. yd.

To conclude our remarks upon roads, we give a short abstract of a paper read before the Institution of Municipal Engineers, which should be of interest to the reader.

The paper by Mr John H. Evans on “Road Management and Repair” dealt with a subject of supreme importance, but about which it becomes increasingly difficult to say anything new. The author, who is the district main road surveyor for the Hertfordshire County Council,* realised this fact, and contented himself with a statement of the results of his own experiences. He has discovered that a scarifier is a tool not required upon roads nowadays, certainly not in rural districts, nor upon 80 per cent. of urban roads. Much more harm than good is done by its use, and the sooner it is discarded the better it will be for the roads, the ratepayer, and the engineer's reputation. The system of new coating adopted by Mr Evans consists of filling up all hollows and uneven parts, and rendering the old surface as even as possible by rolling before the new material is spread. The new material is then applied in the ordinary way, care being taken to see that every shovelful is well and evenly spread and thoroughly dry-rolled until every stone is dovetailed and firmly laid before the binding material is applied. Only enough

* 1909; now, I understand, to the Isle of Man Highways Board.—(AUTHOR.)

The Surveyor to the Renfrew Road Trust, Upper District.

The following may be taken as a fair approximation of the cost of working the steam roller for one week during the average of the weather in summer and winter months, keeping in view the various circumstances above noted:—

		£	s.	d.	£	s.	d.
1 driver	per week	1	7	0			
1 watchman	"	0	18	0			
1 man blinding	"	0	17	0			
2 men for water-butts	"	1	11	0			
3 men sweeping	"	2	11	0			
					7	10	0
2 horses and men for water-butts, at 51s.		5	2	0			
1 horse and man two days blinding, at 8s. 6d.		0	17	0			
					5	19	0
Coal, Oil, packing, etc.					1	0	0
Depreciation and repairs on cost of roller, say 10 per cent. per annum—per week					1	0	0
					£15	9	0

The above cost per week for rolling should produce about 3600 superficial yards of road rolled smooth.

The Wear and Repair of Roads.—On all stone roads water will have a more or less detrimental effect, usually equal to $\frac{1}{4}$ of the total wear, while twice as much will be produced by horses' feet than by the actual wheels of horse-drawn vehicles. Experiments have proved that friction will absorb about $\frac{1}{100}$ of the total dead load, that is to say, for a load of 1 ton = 2240 lbs. the tractive force required to overcome the effects of friction would be 22.4 lbs. As the result of this,

each individual road stone is subject to two forces acting upon it, shown in fig. 407. The horizontal component is inversely proportional to the diameter of the wheel producing it. The use of springs will tend to diminish the effects of the vertical component; their use may also diminish the tractive force required as much as 10 lbs. per ton. They have, however, no effect upon the horizontal component. Dished wheels produce a constant twisting action upon roads. This tends to grind up the road, and they are altogether very heavy on the surface. Cylindrical wheels are much less severe, because their action is that of a simple rolling motion. They will also distribute the load over a wider area by reason of the whole tyre coming flat upon the surface of the metal.

The actual pure legitimate wear of roads is due to them becoming thinner and thinner; in fact, like anything else, they will, if not properly repaired, wear out. Such a state is, of course, never permitted by a good road surveyor; at the same time, such a person is largely in the hands of the overseers if the district be large and sparsely populated. When making up road estimates, it is useful to bear in mind that a mile of road 12 ft. wide and 1 in. thick = 200 solid cub. yds.; this would be equal to about 400 cub. yds. of broken metal. Thus if we go and spread the whole mile of road with 2-in. broken

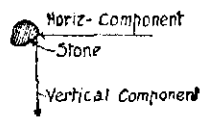


FIG. 407.

metal 1 stone thick (called "sheeting"), we shall require about 800 cub. yds. of metal. The amount of metal which road surveyors take to the cubic yard varies. The author usually assumes 21 cub. ft. = 1 ton. This, of course, will vary greatly, as described under the testing of road stones in the previous chapter.

In repairing ruts in roads, they should, of course, be filled as they appear. To do so, they must be cut square with the pick and refilled with metal to just above the surface level, but not heaped. It may here be stated that the author is of the opinion that dividing roads into small districts under the charge of one man, for all repairs, cleansing, emptying cesspits and gullies, is by far the best way, and then have one overseer whose duty it is to see the men do their work and look after the distribution of the broken metal. Each man should have his own supply and his own tools. By this system a surveyor has a check over his men, and can see who are competent and who are not. By instituting a bonus system the men will try hard to do their best, with immense satisfaction to men, overseer, and surveyor, to say nothing of the ratepayers. Hollows, which usually occur oval in shape, unlike the ruts, must not be cut square, but simply filled up in their ordinary shape. A few stones should be put on and allowed to sink in, then a few more, and so on, in preference to heaping up the bad patches, which only scatters the metal, to the disadvantage of everyone, to say nothing of the waste of metal and detriment to the road surface. Regarding repairs in general,

There are three main ways in which this is usually done, viz.—

1. Without scarifying.
2. Picking up the edges of the bad places only.
3. Complete scarifying.

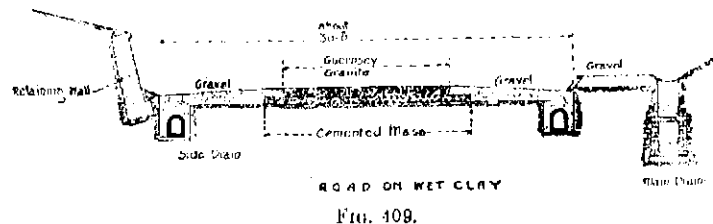
No. 1 is suitable only on roads where the superficial coating is very thin, No. 2 is the general practice in urban and rural districts, while No. 3 has to be resorted to if the road surface is very hard, any binding material being afterwards added.

When patches are laid, it is the prevalent practice to leave them alone to the mercy of the traffic. This, of course, is radically wrong, and is quite avoided if the system suggested is adhered to. They, of course, want the stones which have got loose carefully raking into place. A chessboard arrangement of patching is the most satisfactory, because they will all get their fair share of the traffic. On a steep gradient, however, the best practice is to completely sheet half the road over. The descending vehicles will then tend to use this side as a check on the speed. Autumn and early winter is the proper time for such repairs, but steam rolling may be done at any time. Rolling in patches is very unsatisfactory and wasteful; a thin sheeting is much better. Contrary to the views of some surveyors, the proper cleansing of roads is most essential to their durability, because it will be found that mud on roads consists of quite 57% of horse manure, which, by its *acid* action, is very detrimental. The rest is composed of crushed stone 30% and abraded iron 12%. A fair percentage of rubber is also sometimes found. The more frequently the roads are cleaned, the less total quantity of mud will be produced in the end, and consequently less labour.* It is, however, a bad practice to perform too much scraping on siliceous roads, because they are liable to get very dry, and the stones will work loose, and they want watering well. Sea water may be used on these roads (but on this class of road only). The natural salt is deposited, and its hygroscopic effect, in turn, will retain the

* It is an advantage to remove dust in dry weather before rain converts it into mud.

(and as little as possible) of this material should be used to cement the new surface, after which clean $\frac{1}{2}$ in. chippings should be spread lightly and swept about, the roller being allowed to finish the work. Upon hills it is found necessary to slightly score the surface before spreading the material, in order to prevent slipping. This scoring is done diagonally from the centre to the sides in lines from 12 in. to 18 in. apart. Mr Evans does not believe that tar, or any material of which tar forms an ingredient, is the last word in the cure and prevention of the dust nuisance. Tar has not yet had a fair chance of showing its benefits, but so far it has been tried with indifferent success. When tarring is carried out, it should not be immediately covered with sand or hoggin, which are too fine and absorbent. Nothing but small, clean, dry chippings should be used. Until tar is standardised it cannot be relied on for efficiency. If tar be applied, a pressure of from 50 lbs. to 60 lbs. is ample, and the temperature should only be sufficient to render it fluid for working, say 150 deg. Fahrenheit; actually to boil the tar is a mistake, as it is apt to dry too hard and too quickly before the chippings have become impregnated.

Owing to the fact that detailed cross sections of roadways are not used much in practice, we have not overburdened the reader with many in this chapter. For the benefit, however, of those who are quite ignorant of road



construction generally a few typical illustrations are given, so as the previous chapters may be all the more readily understood and appreciated by him.

In fig. 409 we have a full section of a 30-ft. highway on wet clay, interspersed with springs. The left bank had to be retained by piles. It is liable to slip; two side drains are provided, delivering into a *dry* rubble main drain. It may also be set in mortar if necessary.

Fig. 410 shows another type of road to suit similar conditions, a section through one cesspit being shown. There are two gravel footpaths, and circular brick discharging culverts.

Fig. 411 shows a road on a clayey loam soil, having two side drains, one main discharging drain, and mitre drains at intervals. There are two footpaths 5 ft. wide. All these drawings are fully dimensioned, and are actual examples of practice in the south of England.

Fig. 412 shows part section of a hand-pitched road, of the form now usually adopted to macadam surfaces.

Fig. 413 shows a paved road with a concrete foundation.

Fig. 414 shows a paved road with a pinned foundation.

Fig. 415, a wood pavement with an asphalt expansion joint.

Further typical examples of municipal roads are given in figs. 415A, B, C, D.

Asphalt Footpaths.—Preparation and laying tar paving. Gas-tar should not be used until it has been stored for at least six months, in order that the volatile oils and any water that may be in the tar may escape and come to the surface; and the tar should be drawn from a covered tank at least 10 ft.

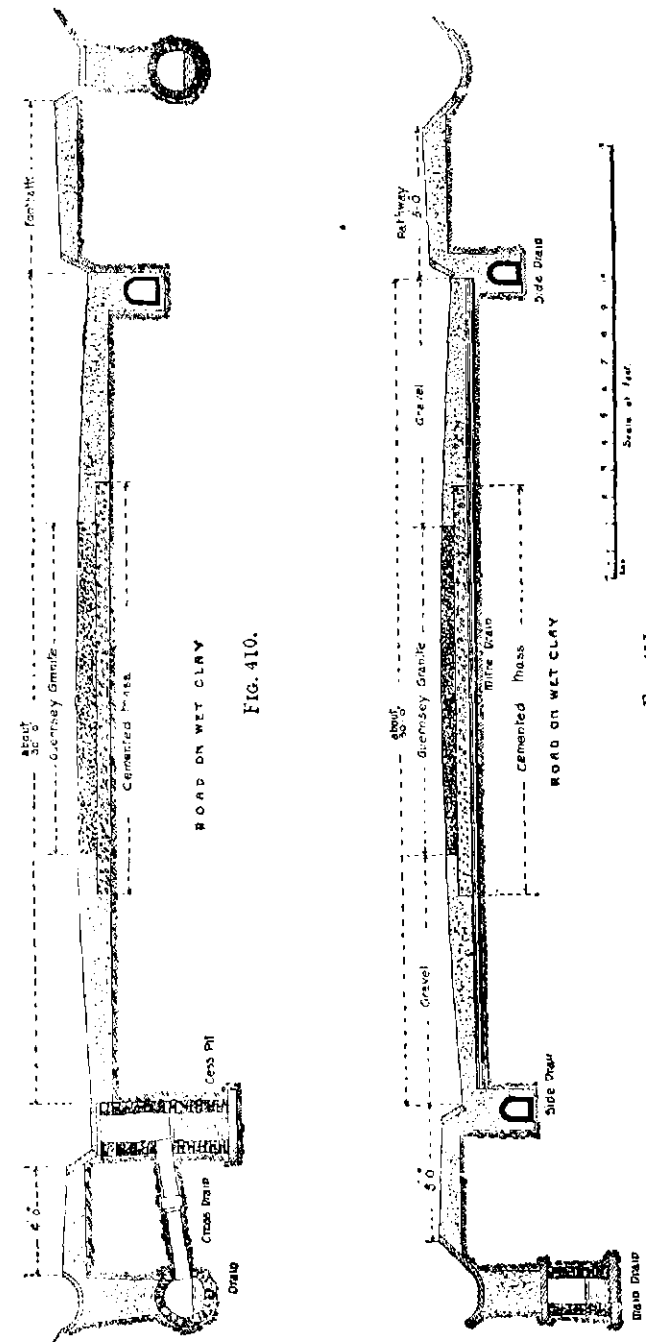


FIG. 410.

FIG. 411.

deep. The stones that are used should be fairly uniform in size, and for the bottom layer should not exceed 1 in. in diameter, and should be retained by a $\frac{5}{8}$ -in. mesh sieve. The stones for the top layer should pass through a $\frac{1}{2}$ -in.

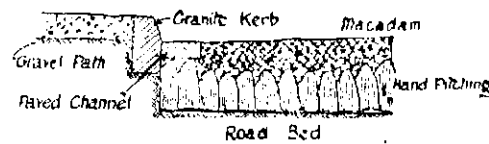


FIG. 412.

sieve, and be retained by a $\frac{1}{2}$ -in. mesh sieve. Derbyshire stone or Kentish rag produces the best tar paving. The stone is heated best on a hot hearth or an iron floor, below which runs the flues from the fire, the stone being raised to such a temperature that it will absorb as much tar as possible without ignition. Hot tar is added to the heated stone, and the whole turned over with hot shovels until all the stones are well coated. The quantities required are 12 gallons tar, $\frac{1}{2}$ cwt. pitch, and 2 gallons creosote, to 1 ton stone. The paving is best laid in the spring or winter. The foundation should consist of hard brick rubble, furnace clinkers, or similar hard core,

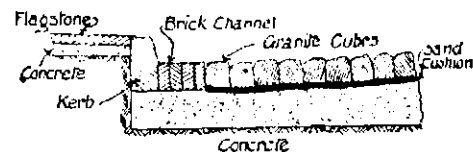


FIG. 413.

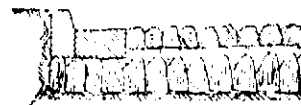


FIG. 414.

broken to pass a 3-in. ring, levelled and thoroughly rammed for a depth of 4 in. The first layer of tar paving should be $1\frac{1}{2}$ in. in thickness, the top layer 1 in. thick. Both layers to be well rolled with a water-ballasted roller, 10 cwt. The surface should be well gritted with Derbyshire spar grit, stone dust, or sharp sand. Tar-macadam should be composed of two coats, $2\frac{1}{2}$ in. thick of uniform material, free from dirt, broken uniformly to a $1\frac{1}{2}$ -in. gauge. The third coat to be 1 in. in thickness, of 1-in. slag, shingle broken to a $\frac{3}{4}$ -in. and $\frac{1}{2}$ -in. gauge, in equal proportions, free from dust. The total thickness, after being well rolled, should be 5 in., and each layer should be well rolled before the next is put on. The top of the surface should be $\frac{3}{4}$ in. above the level of the channels. After rolling, the roadway should be covered with a dressing coat of granite or limestone chips, and again rolled. The road bed should be perfectly dry before the tar-macadam is laid, otherwise it may prove a failure. The surface requires to be tar-painted every three or four years. Quantity of materials required: One barrel tar, 1 cwt. pitch, 4 gallons creosote, to 1 ton stone. Asphalt footpaths.—Asphalt should be laid on a perfectly solid foundation, either of 6 in. of good stone or gravel, thoroughly consolidated by rolling, or of 4 in. of Portland cement concrete. No asphalt should be laid on the concrete until it is thoroughly dry. Mastic asphalt should be used for footpaths. The usual preparation of the mastic is as follows: According to the amount of bitumen contained in the natural stone,

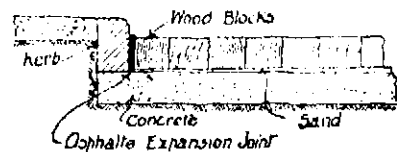


FIG. 415.

BACK ROADS

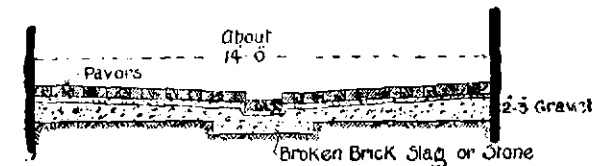


FIG. 415A.

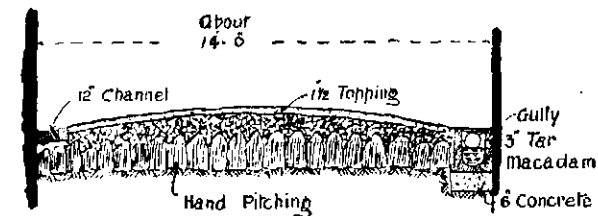


FIG. 415B.

SUBURBAN ROADS

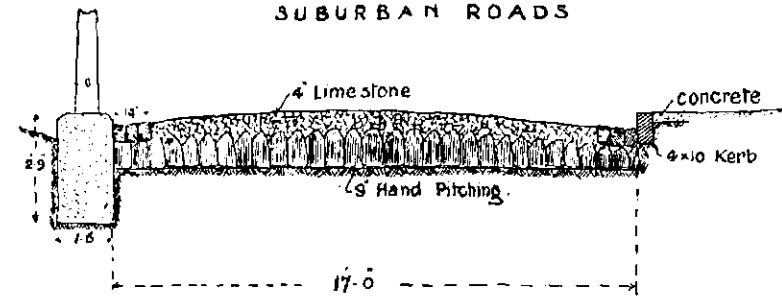
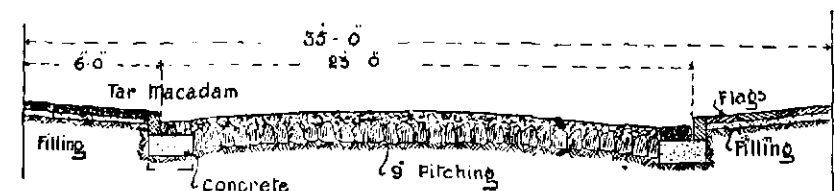


FIG. 415C.



U.D.C. ROAD

FIG. 415D.

from 5 to 8 per cent. of bitumen is placed in a large cauldron, which is usually provided with agitators, driven by steam power. When this is thoroughly melted, the powdered asphalt is added little by little, the heat being raised to between 390 degrees to 480 degrees Fahr., the mixture kept well stirred and cooked for about five hours. It is then turned out into iron moulds, and about 20 to 60 per cent. of grit added, according to the nature of the work. Mastic asphalt is sent to different parts of the country, prepared at the works in cakes, which need only be melted down in ordinary cauldrons and laid in ordinary manner. A small quantity of bitumen is added in this case to make up for loss by evaporation. Quantities of asphalt, bitumen, and grit required: 13 cwt. pure block mastic, 12 lbs. bitumen, 7 cwt. grit or washed sand.

On the subject of roads, much valuable information is now at hand as a result of the International Road Congress which took place in 1908 in Paris, due to M. Barthou, Minister of Public Works in France. A very concise report has been prepared by Dr P. C. Cowan,* Chief Engineer to the Local Government Board in Ireland, of which the following is an abstract. The principal matters discussed were:—

- (1) *The present road.*
Bed, choice of coating, method of execution, cost, critical examination.
- (2) *General modes of maintenance.*
Metalled highways, paved highways, various highways.
- (3) *Struggle against wear and dust.*
Cleaning and watering, use of tar, use of various products, technical and economical results.
- (4) *The future road.*
Direction, longitudinal and cross sections, coatings, curves, various obstacles, special tracks, etc.
- (5) *The effects of new modes of traffic on roads.*
Damage due to speed, damage due to weight. The influence of the tyres, of anti-skid devices, etc.
- (6) *Effects of the roads on vehicles.*
Deterioration of parts, side-slip, etc.
- (7) *Road signals.*
Milestones, indication of direction, distance, height, obstacles, dangerous points, etc.
- (8) *The road and mechanical transport.*
Public carriages, industrial carriages, tramways.

QUESTION 1.—*The Present Road.*

It was agreed that—

(a) The foundations of a road are of primary importance, and should be carefully adapted to the nature of the subsoil and the probable traffic. For all paving a concrete foundation 4 to 5 in. thick on a thin layer of sand is to be recommended.

(b) Trials as to the use of bituminous compounds for road purposes should be continued and extended in order to arrive at economy and efficiency.

(c) A suitable binding material should be used in minimum quantity during rolling.

* The author is much indebted to Dr Cowan for permission to reproduce the abstracts that follow. The full report is on sale at Messrs Ponsonby & Sons, Grafton Street, Dublin, price 2d.

Regarding foundations, really very few roads have any in the true sense of the word, especially in Ireland and Scotland; and in fact, provided the crust of the road is thick and strong, and the soil of a porous nature, the consequences are not by any means disastrous, and to rip up an old road simply to provide a foundation would not be a wise or economical procedure. With a heavy retentive soil, however, it would be necessary to make the road really efficient. All paved roads, however, must have foundation.

QUESTION 2.—*General Modes of Maintenance.*

It was agreed that—

(a) Darning or repairing road surfaces by small patches should be restricted as much as possible, and confined to keeping the road smooth, pending sheeting with the roller.

(b) As far as possible, only hard homogeneous road materials regularly broken should be used.

(c) The whole width of a road should be resurfaced at once wherever it is possible to divert the traffic, and notice boards should be set up at junctions of roads to enable the traffic to avoid the road under repair.

(d) Authority to place gas and water pipes under paved roadways should only be given in exceptional cases, when there is no other practical solution.

The general conclusion arrived at is that for comfort and economy repairs must be promptly executed, and it is well to fix patches by rolling.

QUESTION 3.—*The Struggle against Wear and Dust.*

It was agreed—

(a) To recommend the use of suitable paving or other improved surface as a remedy for wear and tear, as well as dust upon roads subjected to traffic heavy in character or in weight.

(b) To recommend cleaning and frequent light watering by mechanical means, and the use of surfaces which can be easily cleaned.

(c) That emulsions of tar or oils, hygroscopic salts, etc., are really efficient, but unfortunately only for a short time. Their usefulness is therefore limited to special occasions, such as festivals. It is, however, advisable that further experiments should be made with such materials. The placing of trees along the roads is worthy of encouragement from the point of view of the suppression of dust.

(d) That tarring, when well carried out, is undoubtedly an efficient remedy against dust, and also to some extent protects the roads from the destructive action of vehicles in general, and fast motor cars in particular.

(e) That experiments up to the present with tar incorporated in the road materials are not yet sufficient to allow a definite judgment to be passed on the results obtained, and that these experiments should be continued with due regard to the experience acquired in different countries.

In considering the dust problem, it must be borne in mind that there are two main causes for dust.

1. Dust generated in the road itself by wear.
2. Dust carried on to the road, which can only be reduced by frequent watering and sweeping.

Mr Maybury, county surveyor of Kent, a county in which there is an immense amount of traction-engine traffic, expressed views in close agreement with those already noted. He states that tarred macadam roads are by no means dustless, and require tar-painting each year just as though untarred

material had been used. In his opinion, the outcome of experiment and experience, a sure and certain way of improving and fitting the roads for the new traffic is—

(a) To build them up as strongly as possible, reducing the camber to one-fiftieth of the width uniformly.

(b) To coat the surface with the best obtainable hard material, 2 in. to 2½ in. in size, well rolled with hard gravel and chippings as binding.

(c) To clean the surface and apply a dressing of heated tar compound, covering same with clean gravel or chippings, and rolling well with a steam roller.

Dr Cowan says:—I may here note that modern engineers are evidently somewhat sharply divided as to the relative merits of surface tarring and tar-macadam, and there is a distinct difference of opinion as to whether in tar-macadam the stones should be of the same size or of different sizes. The most reasonable view appears to be that they should all be of moderate size, between 1-in. and 2-in. gauge.

There was a consensus of opinion that coal-tar is most advantageously applied hot, and that the surface should then be sprinkled with fine stone chippings. Tar can only be applied to a dry road in fine weather, and it is useless to apply it to a road surface which is not in good smooth order and swept quite clean. Oil-tar, which is a by-product from the enrichment of water gas, is much more fluid than coal-tar, and is an excellent dust layer. It has been used in Rathmines and Pembroke,* and has the advantage that it does not require heating or covering with screenings or sand. Mr Reginald Lambton, an English surveyor, states that, from extended experiments, it has been found that four dressings of oil-tar are required each watering season, the total cost of which is about one penny per sq. yd. At the Congress little mention was made of this kind of tar, the annual output of which in the United Kingdom is about 18 million gallons, as compared with about 180 million gallons of coal-tar.

QUESTION 4.—*The Future Road.*

It was agreed that—

(a) Where the traffic of self-propelled vehicles is not very great, the present road, if constructed and maintained in accordance with the resolutions passed on the first two questions, is satisfactory.

(b) The carriage-way of the road of the future should be homogeneous, and composed of materials which are hard, tough, resistant, and non-slippery.

(c) One roadway should be provided for all vehicular traffic, 6 metres (say 20 ft.) wide at least, save in the case of park avenues, where several separate roadways are desirable.

(d) The road should have the least camber compatible with the easy off-run of water.

(e) Curves should have a radius of at least 50 metres (say 164 ft.), and to avoid sharp curves an easy gradient may be sacrificed.

(f) The roadsides should be properly raised on the outer side of curves, but not to an extent inconvenient to ordinary traffic. A narrow footpath should be provided on the inner side of the curve, and no deposit of heaps of stones allowed at curves on roads.

(g) Road crossings should be visible and well opened out.

* Suburbs of the city of Dublin, but not under the Corporation.

QUESTION 7.—*Road Signals.*

It was agreed that it is desirable—

(a) That a general and uniform plan of marking distances should be adopted, with milestones of uniform pattern, showing the distance from large towns in such a way as to facilitate the calculation of cumulative distances.

(b) That warning signals should graphically indicate the kind of danger they refer to, and should be limited to the following cases:—

- (1) Obstruction on the road.
- (2) Corner or sharp turning.
- (3) Level-crossing (railway or tramway).
- (4) Dangerous cross-roads.

(c) That danger signals supplied and erected by private bodies with the approval of the authorities should be protected by law.

QUESTION 8.—*The Road and Mechanical Transport.*

It was agreed that—

(a) Automobile vehicles may be advantageously used for public conveyance, without injuring the road to any noticeable extent, if the average speed does not exceed 18 kilometres (say 11 miles) per hour, and the maximum speed does not exceed 25 kilometres (say 15½ miles) per hour. The weight on the heaviest axle must not exceed 4 tons, and the pressure on each centimetre of width of tyre must not exceed 150 kilogrammes (i.e. 825 lbs. per in.) for wheels of the diameter at present in use.

(b) Internal-explosion motor-lorries may cause no injury to the road, with an average speed of 10 kilometres (6¼ miles) per hour, and a maximum speed of 15 kilometres (9½ miles) per hour, with maximum axle load of 5 tons and smooth metallic tyres on the driving wheels, and with a load not exceeding 150 kilogrammes per centimetre (825 lbs. per inch) of width of tyre.

(c) It is difficult in the present condition of the roads and of the automobile industry to answer the questions arising from the traffic of heavy steam-lorries and traction-engines: as their use is necessarily limited to a comparatively small extent, it would be useful, in case of need, to fix definite routes on existing roadways.

(d) It is desirable to collect exact particulars from competent authorities to determine the proper relation between the construction of roads and road structures, and the speed, weight, width, and nature of tyres, diameter of wheels, and method of suspension of wheels, wheel-base, and number of axles of vehicles.

(e) Tramways should be laid clear of the ordinary roadway, or on special tracks leaving a minimum width clear of the track of 5 metres (say 16½ ft.).

(f) If tramways are laid in the roadway, a clear width of at least 2·60 metres (say 8½ ft.) should be left, and the tramway rails should have counter-rails, either separate or combined with the main rail.

The resolutions as to weight on tyres omit any consideration of the diameter of the wheels, which is an element of the very greatest importance, and the English-speaking members passed the following distinct resolution:—

“That heavy self-propelled vehicles and their trucks should be regulated as to speed, construction, width of tyres, and the diameter of wheels on the general basis of the Heavy Motor Car Order, England, 1904, but that the maximum weight per in. width of tyre should not exceed 600 lbs. on the basis of a wheel of 3 ft. diameter.”

seemed to warrant the opinion that the advantages of the direct labour system far outweigh its drawbacks, and the tendency is decidedly towards its further extension.

Mr Westropp Bennett, of the Limerick Co. Council, said that in the working out of direct labour schemes whole bodies of labourers had control of districts and localities through their organisations, and that deterred councils from carrying forward what would appear to be the best method of organising direct labour on roads.

Mr Collen, the Co. Dublin Surveyor, congratulated Mr Leebody, the County Surveyor of South Tyrone, on his paper on "Econometrical Supply of Road Metal," which he said would be very useful to surveyors. The principle of councils owning quarries and selling the stone to contractors was extremely helpful and beneficial to small contractors, whom he was sorry to see disappearing. He proposed the following resolution for the consideration of the committee:—

It is desirable that all quarries from which stone is obtained for the repair of public roads be owned and worked or leased by the County Councils.

Mr Mulvaney, Co. Surveyor of Roscommon, thought it was desirable to work by direct labour, as the engineer-in-charge was able to effect the works he knew to be absolutely essential to good results. If they wanted to have a high standard of road surface, as called for by modern locomotion, they must adopt a direct labour system.

Mr Hackett agreed as to the advantage of direct labour.

Mr M. E. Ennis, J.P., a member of Wexford County Council, contributed a paper on trunk roads. He asked why Irish roads did not compare favourably with English, Scotch, or Welsh roads. The cause might be traced to the fact that the average cost of maintenance per mile of rural roads is in England and Wales £32 per mile, Scotland £26, and Ireland £14; but the burden upon the rates imposed by this expenditure is, in England and Wales 4½d. in the £1, in Scotland 4¾d., and in Ireland 1s. Scotland and Ireland, with nearly similar areas, 19,069,770 and 20,327,947 acres respectively, and with practically similar populations, 4,472,103 and 4,458,775, expend almost identical sums annually upon road maintenance, the figures being Scotland £1,249,266, Ireland £1,140,626; but the mileage of roads maintained shows a striking discrepancy, as Scotland for her £1,249,266 upkeeps 24,615 miles, whilst Ireland for her slightly lower expenditure of £1,140,626 upkeeps 54,064 miles. With regard to trunk roads, he said he regarded a trunk road as a national highway or great trade artery of uniform type and high grade, controlled and maintained by a central authority or by a committee of association of local authorities. He strongly advocated joint action on the part of authorities in Ireland with a view to the provision of a system of national highways, based upon a carefully laid plan, not too ambitious in its inception, but capable of wide future development. Such roads should be capable of carrying heavy traffic, should be well surfaced, and steam rolled.

Economic.—Ireland is a poor country and cannot afford a high expenditure on its public works, consequently even a low price per mile of roads results in a very considerable poundage. This is very clearly shown in the annexed table, which is taken from a paper read before the Institute of Civil Engineers, Ireland, by Dr Cowan, Chief Engineering Inspector, Local Government Board:—

Country.	Total Mileage of all Rural Public Roads.	Cost of Maintenance.	Cost per Mile.	Gross Valuation of Country per Mile of Road.	Total Acreage of Land per Mile of Road.	Average Poundage Rate to raise Cost of Maintenance.
<i>England and Wales.</i> Rateable value, 99,355,590 (1904)	119,037	£ 3,767,977	£ 32	£ 1675	314	4½d.
<i>Scotland.</i> Rateable value, 10,144,822 (1903)	22,542	£ 576,179	£ 26	£ 1338	845	4¾d.
<i>Ireland.</i> Rateable value, 5,201,481 (1904)	54,064	£ 779,174	£ 14	£ 280	376	1s.

The divergence of opinion on this question is perhaps more apparent than real; and if time had permitted, a fuller discussion would almost certainly have resulted in a general agreement.

The Irish Road Congress (1910) also produced evidence from engineers whose experience with roads is very considerable, and which is of the utmost value to municipal engineers.

The percentage of steam-rolled roads in Ireland is comparatively small, and in many districts surveyors have to work at a great disadvantage, due to lack of money; in fact, although many of the roads are in a deplorable condition, yet, putting all circumstances together, the roads as a whole reflect great credit on Irish county surveyors, the worst roads being actually found in parts of Dublin itself. One of the great disadvantages under which Irish county surveyors work is shown by the words of R. J. Meeredy, Esq.:—

"It may be urged that the county surveyor should insist on the new contractor maintaining the road in the condition in which he received it. Theoretically this is quite correct, but the difficulties in the way of the county surveyor are enormous. In some cases his council is hostile, and is inclined to favour the small contractor, who is generally a neighbour possessing considerable interest amongst them. The councillors therefore will not support the county surveyor when he defers or reduces payment, and a lot of claptrap is talked about the roads being good enough for the farmers, and the ratepayers' money being spent in making avenues for motorists and cyclists. Even when the county surveyor has a freer hand, the legal process is a complicated, protracted, and troublesome one, which he naturally hesitates to put in action."*

The main points of interest to the municipal engineer set forth in the above Congress are as follows:—

Steam Rolling of Roads.—Mr E. A. Hackett, M.Inst.C.E., County Surveyor, County Tipperary, S.R., read a paper on this subject. He observed that there are very many desirable methods of improving Irish roads. Of all these, steam rolling is the most effective, and not merely so, but far and away outstrips in importance every other proposed improvement. It is immediate in effect, in many cases gives immediate relief to the rates, and is universally appreciated. It greatly increases the comfort and ease of travelling, enables 25 per cent. more loading to be carried and to greater distance, saves wear and tear on horses and equipment, prevents sore feet in cattle, and is a blessing to farmers, cyclists, and motorists. The heavier rollers are the most economical to work, but for narrow streets you must have a roller which has a narrower roller-path, 6 to 10 tons, say. Summer work is always the best. Roll until the stones present a smooth surface, but not until the divisions between them are obliterated, as if that be done the stone gets crushed. If necessary, scarify road into proper shape before spreading stones, and coat with a layer that, when complete, gives 3 inches in thickness. The supply and spreading of stones may be done by contract, but it is more successfully done by direct labour.

Mr J. F. Beatty, B.E., A.M.I.C.E., Assistant Surveyor, County Antrim, also read a paper on steam rolling, especially with regard to the working of that system in County Antrim, where five steam rollers are at present at work. These five rollers have been kept constantly at work since their arrival, except during frosty weather, and when being overhauled or undergoing repairs; the actual tonnage of broken stones rolled in has averaged

* *The Irish Times.*

annually for the 10-ton roller 9000 tons, for the 12½-ton roller 10,500 tons, and for the 15-ton roller 12,000 tons, the stones being rolled in at the rate of about 3½ tons per hour with 10-ton roller, 4½ tons per hour with 12½-ton roller, and 5½ tons per hour with 15-ton roller. The cost of rolling per ton, therefore, to the various districts using the rollers, works out at 1s. 3½d., 1s., and 10d., for the 10-, 12½-, and 15-ton rollers respectively, based on the chargeability of 4s. 6d. per hour for the time worked by each roller; the actual cost per ton for rolling done up to date has been considerably less.

The Economy of Good Stone.—Mr Wm. Collen, M.Inst.C.E., County Surveyor of Dublin, in the course of his paper on the above subject, said:—The quality of a stone used in the surface of a road should be suited to the traffic it is subjected to, i.e. a hard and tough stone should be used on a road over which heavy traffic is carried continuously; for light traffic, a comparatively soft stone, i.e. one of a different mineral character, will generally prove sufficient. If, however, a hard stone abounds in the vicinity of a road it should be used whether the traffic is heavy or light, i.e. a soft stone should not be brought from a distance to a road subjected to light traffic, for it may thus become as dear as the hard stone, and, apart from the influence of the traffic upon it, will not wear as well or last as long. The converse of the last statement, however, contains a greater truth—for it will be found that the greater cost is more apparent than real. In other words, if the period of time through which the stone surface survives, and the labour expended in cleaning and repairing it, are taken into account, it will be found the hard stone is generally in the end the cheaper. It is estimated that quartzite will last three times as long as limestone, and will require about one-fourth only of the labour in cleaning. There should therefore result in the use of this stone on the road a saving of about 60 per cent. after three years. To the users of the road, too, there will be a distinct gain in the use of the better material, requiring, as it will, to be rolled less often, and causing therefore less interruption to the traffic.

Repair of Roads.—Mr J. S. Pickering, M.Inst.C.E., Borough Engineer, Cheltenham, in dealing with this subject, said he had long been of the opinion that the systematic repair of roads by patching does not receive the attention its importance demands; and simple as the operation is, and notwithstanding its economy, it is not generally carried out in a scientific manner. Some authorities appear to be satisfied with repairing holes, which become a source of danger, while others attempt patching by filling up the inequalities with loose material, leaving the traffic to either consolidate it or to scatter it about, according to the state of the weather and the condition of the roads. The author's contention is that the patching of a road should be carried out immediately it begins to show any appreciable signs of wear. The result of such a practice is, undoubtedly, not only to maintain a better surface, but to lengthen the periods for recoating, and consequently to reduce the cost of maintenance. The author came to the conclusion that what was required was a special roller to be used almost entirely for patching purposes. Such a roller, it was thought, should be as capable of rolling in a patch and leaving it quite uniform with the adjoining surface. It should be small and compact, in order to be as little conspicuous in the streets as possible. It should be capable of being quickly and easily reversed, in order that small patches could be consolidated quickly.

Mr D. Megaw, B.A., B.E., supplied notes on the working of three schemes of direct labour in operation in County Antrim, the experience of which

CHAPTER XX.

THE DESIGN OF ENGINEERING STRUCTURES
IN CONCRETE STEEL.

THE design and erection of large structures in reinforced concrete, or, as it is perhaps more appropriately termed, concrete steel, is attracting the attention of nearly every civil engineer of the present day, both European and American. The enormous possibilities of the art are only as yet merely touched upon, and very little understood. Concrete steel must not be confused with concrete used together with steel joists and sections for large fireproof buildings. In this chapter we are going to consider how steel, intimately "blended," so to speak, with the concrete in a structure, is going to relieve that concrete of the greater part of its tensile stress, with resulting economy and safety in the structure. The use of rolled-steel sections used in Steel Construction (Chapter XXXIV.) is practically entirely eliminated by rods and bars of a comparatively small sectional area. These are always bedded in the concrete, and are invisible to the eye. The uses of concrete steel would have numerous applications, and sometimes odd ones too. Bridges, large buildings, reservoirs, tanks, water-towers, canal locks, sewers, culverts, are all common applications of the art, to say nothing of barges, telegraph poles, gates and boundary-fences, etc., which have been tried in actual practice, and given satisfactory and economical results.

The first thing to do is to investigate the essential qualities of the two ingredients, viz. concrete and steel. Concrete itself was considered in Chapter XIII., but as it is of special importance in this chapter we shall say something more about it.

The Aggregate.—When ballast is used it should be well washed and quite free from loam. It should contain a fair percentage of sand. Broken limestone makes a very good aggregate. Its disadvantage is that it cannot be considered as fireproof, the lime being subject to calcination at high temperatures. Sandstone is not to be recommended. Those crystalline rocks called *Diorite* are excellent (Chapter XVIII.), as also is crusted granite. Broken brick can be well recommended. It has great fire-resisting qualities, and will not splinter at high temperatures. It must, however, be very well washed before use, and used wet. Blast-furnace slag will be found satisfactory if it is selected with great care. It must, however, be porous, and not too glassy, and absolutely free from sulphur. The presence of sulphur in any aggregate must be entirely eliminated, and is only done so by repeated washings and weathering.

Sand.—The sand used in this particular work requires very careful selection. In the first place, it must be a happy medium between dust and coarse gravelly

sand. It must be clean. Regarding the fireproof qualities of aggregate, furnace slag appears to have given unsatisfactory results. Broken brick and granite gave good results, coke breeze and furnace clinker gave the best.

When concrete is used for walls, $\frac{3}{4}$ -in. aggregate is ample. It is the sharp sand which is the best in practice, because it is usually of a siliceous nature. Test mixtures of the concrete should always be made in the office. Proceed as follows:—

Procure a piece of iron tube and make a mixture similar to that which we are proposing to use. Very carefully note down its composition, then mix it and ram it into the piece of tube, which was previously sealed at one end by means of a screwed cap. Carefully measure its height in the tube when fully rammed. Then clean out the tube again and make a fresh mixture, varying the proportions of sand and broken stone, but still having the same total weight of the two combined. If less or more, it will be a guide to the future adopted proportions. Again note the height of the sample when rammed. Repeat this process a few times. The lowest height proves the best mixture, but they must be very accurately proportioned if they are to be of any practical value. Now, for a mixture of concrete the following proportions, according to the class of work, have been found to answer well in practice:—

	Cement.	Sand.	Aggregate.
Reinforced concrete	1	2.0	4
Arches, ordinary walls, and stairs	1	2.5	5
Heavy walls, piers, etc.	1	3.0	6
Large masses, such as harbour works	1	4.0	8

The aggregate will, however, shrink during mixing. For example, say we have $4\frac{1}{2}$ cub. ft. of Portland cement, 9 of sand, and 20.4 of aggregate. The resulting mass, which might be supposed would yield 34 cub. ft. of concrete, will only yield in practice about 21.4 cub. ft. in mass.

The use of salt water in concrete works has the tendency of retarding the setting power. All water used should be carefully selected; also it must be clean, free from acid and strong alkalies. It must not have much lime present, and be free from organic impurities. The average quantity required would be from 21-24 gallons per cub. yd. of dry mixture.

Concrete should not be tipped from a height, as might be considered an advantage, nor should it be done in frosty weather. If, however, this should be quite unavoidable, as in the case of cold climates, 1% of salt may be added with advantage, or a warm mixture used. The average composition of a cub. yd. of concrete would be as follows, according to the proportions used to make 1 cub. yd. of concrete:—

Proportion.	P. Cement in Lbs.	Cub. Yds. of Sand.	Cub. Yds. of Ballast or Stone.
1-2-4	590	.44	.88
1-2½-5	485	.45	.91
1-3-6	414	.46	.93
1-4-8	320	.48	.96

Regarding forms for concrete work, any cheap wood will do. It should not be seasoned. They must be well coated with soft soap or crude oil before use, to prevent them sticking. In case, however, the particular work has to be afterwards plastered, this will not do; in such a case they must *only* be wetted. Cement mortar may have the proportions of 1-2, 1-3, or 1-4, according to circumstances. The addition of $\frac{1}{2}$ to 1% of lime putty is a distinct advantage, especially for plastering work, as it will tend to make the mortar flow easily, and cause it to be impervious and very adhesive.

All steel used in concrete steelwork must be surrounded by at least 1-2 in. of concrete, according to the size of the structure. Concrete which is made from cinders, as we before stated, gave good fire-resisting qualities. It has, however, the objection when used in conjunction with steel that the free sulphur which it is liable to contain would be likely to corrode the steel. The whole success of concrete steelwork, apart from the scientific design, depends on proper mixing of the concrete. This may be hand done, but when it is, it wants very careful doing by willing workmen. The machine mixer is a much more satisfactory appliance, and is now used on all contracts of any magnitude. They may be hand or power driven, of the rotary type. Taylor's or Ransome's are perhaps the best known as giving satisfactory results. Gravel mixers are also in vogue. The principle is, that by means of battle-plates the mixture thrown in at the top descends by a devious path from one side of the machine to the other, water in suitable quantities being added meanwhile. A gain of about 11% on machine mixing can be relied upon. It must always be borne in mind, however, that gravel concrete is only 88% as strong as that made from crushed granite.

Regarding the strength of concrete in tension, it may be taken anywhere between 200 and 300 lbs. per sq. in., principally according to age. For instance, it has been found by experiment that a 1-2-4 mixture will give the following average results:—

26 days old	280	} tensile strength, lbs. per sq. in.
28 " "	360	
33 " "	305	
35 " "	300	

Designing Beams.—The modulus of rupture of a concrete beam may be taken as—

$$1.666 \frac{Wl}{bd^2} \quad (244)$$

where W = total load; l , b , and d = length, breadth, and depth in inches. This will give inch-lbs. The modulus of elasticity of concrete = E , in tension = 2,000,000 lbs., and in compression 2,700,000 lbs. per sq. in. This is an important factor, because the greater the alteration of the concrete due to the load, the more load will consequently come upon the steel.

Again, we have to consider the coefficient of expansion of concrete. It may be taken as .0000055.

We must now consider the steel used in concrete steelwork. In Chapter XIII, we also made some remarks thereon. The ultimate strength of the steel will vary according to the amount of carbon present. A few examples will serve to illustrate our assumption. They are based upon Baushinger's experiments.

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Percentage of Carbon.	Ultimate Tension.	Elastic Limit in Compression.	Elastic Limit in Tension.
.14	62,945	41,954	39,536
.19	68,096	47,084	43,008
.54	79,072	49,618	48,942
.55	80,412	46,994	49,771
.78	92,064	53,312	53,645
.80	102,816	57,008	63,187
.87	104,608	61,017	56,012
.96	118,048	69,216	71,126

In all cases the shear may be taken as 77% of these values, the usual practical values taken being—

Tensile strength.	75,000 lbs. per sq. in. (ultimate).
Shear strength.	50,000 " "

Now on steel we have to consider the effects of high temperatures. The tensile strength of steel will gradually diminish between the limits of -4° and 150° Fahr. An increase will then take place up to 450° . It then, however, will rapidly fall. Alternation of stress (*vide* Strength of Materials) will have the usual effects, of course, of causing the steel to become brittle and useless. For comparison with concrete, the coefficient of expansion of steel may be set down as .0000066.

Now, from what we have previously said, we can infer that in practice concrete is about ten times stronger in compression than in tension. But its adhesion to steel may be taken as anything up to 600 lbs. per sq. in. This, however, is too high for practical consideration. The average practical limit may be set at 250 lbs. per sq. in. Now, of course, we have to make this bond between the concrete and the steel of greater tenacity than that of the bar, to effect which a certain proportion of the bar will have to be embedded in the concrete apart from practical considerations. To find such a length in inches we have a formula based on our previous reasoning. If F_s = tensile strength of the steel, d = diameter of the rod in inches, f = 250.

As adhesion strength,

$$l = \frac{F_s}{4f} \times d \quad (245)$$

We know that the object of the reinforcement is to reduce the tensile stresses, but it will also be capable of taking some of the shear too; and it must be borne in mind that the greater value of E of the materials, the less will be the deformation under a given load. Then assuming thorough bond between the concrete and steel, the permanent set of the whole beam must be equal everywhere. But, however, owing to the permanent deformation being greater in the concrete than in the steel, we will have induced stresses therein, and hence a compressive stress in the concrete. Now the ratio of " E " in concrete and " E " in steel may be set down as 10. Then, considering a load applied to a reinforced concrete beam, assume the bond to be perfect. But we know that steel will elongate more than the concrete; the result then is that the load gradually comes off the concrete and goes on to the steel. In fact, we may safely conclude that the behaviour of concrete steel under load is

invariably governed by the ratios of "E" of the two materials. Such is the theory of structures of concrete steel. We shall consider now the action design of beams connected with such structures.

In concrete steel design it is not usual to allow any tensile strength in the concrete. It will, however, extend proportionately with the steel. The consequence of this is a series of minute cracks in the concrete; these, however, are invisible to the naked eye; their presence, however, destroys any tensile value the concrete might have, and hence its omission in calculations. Now in the previous theory we pointed out that the values of "E" in compression and E in tension have different values. The result is that in any simple concrete beam of rectangular section the neutral axis will *not* lie in the centre of that beam; this is also the case in concrete steel beams. Now in an ordinary rectangular beam, if f = the extreme fibre stress, w = total load in tons, and b , d , and l in inches,

$$f = \frac{75Wl}{bd^2} \quad (246)$$

This equation assumes the two values of E referred to to be equal. Therefore its use in our present consideration would give a value too high or too low in some cases, and is consequently of little value here. The first thing to do is to find the neutral axis of our proposed beam. Take fig. 416 aa = the neutral axis, bb = axis of compression, and cc = axis of tension, the latter we assume to pass through the centre of reinforcement, in both cases of the distance from the neutral axis to the extreme fibres. Then if we neglect tension in concrete, and also the fact that the beam will tend to alter its shape due to bending, the total deformation of the steel will be equal to that of the concrete. Now we have to establish a ratio between the values of E in compression and E in tension—call it M. The calling f_t the extreme fibre stress in tension and f_c the same in compression we have a certain quantity which we will call Z; its value is

$$Z = \frac{1}{1 + \frac{f_t}{f_c} \times M} \quad (247)$$

From which reasoning we conclude that the position of the neutral axis of concrete steel beam is directly affected by the resistance to tension and compression and the two coefficients of elasticity of the materials in question.

Now in all beams we have a bending moment which must have an equal and opposite resisting moment, and the

$$BM = \frac{Wl}{8} \quad (248)$$

(vide Beams), while

$$RM = \frac{fI}{y} \quad (249)$$

I being the moment of inertia of the section, which for a rectangle is

$$\frac{bd^3}{12} \quad (250)$$

and y = distance from neutral axis to the extreme fibres.

$$\text{Again} \quad RM = fby^2y \quad (251)$$

$$\text{and transposing} \quad W = \frac{5.33fby^2}{l} \quad (252)$$

Or we may want to find the extreme fibre stress in the top or bottom of our beam. Its value

$$= f = \frac{Wl}{5.33by^2} \quad (253)$$

this being usually taken as lying on the centre of the reinforcement. All these equations assume a uniform cross section; they are reduced one-half when W refers to a *centrally* applied load. In all beam designs we shall take a permissible unit stress in the steel = 15,000 lbs. per sq. in., and 500 lbs. per sq. in. in the concrete (in compression). If any tension be considered at all, 50 lbs. per sq. in. may be taken. Now up to certain limits the higher we place the neutral axis of the beam, and the lower the steel reinforcement, the greater the advantage of our reinforcement.

Coming to the actual calculations necessary to design any beam in concrete steel, to fulfil certain conditions we must have—

The span in inches = l ,

The proposed breadth of the beam in inches = b ,

w = load, including weight of beam in lbs. per foot-run. Then if d = depth of beam in inches,

$$d = \sqrt{\frac{wl^2}{600b}} \quad (254)$$

$\therefore b \times d$ = required sectional area.

And the sectional area of steel required

$$= .006bl \quad (255)$$

Say we want a beam over a 10-foot span to carry a total load of 1 ton = 2 cwt. per foot-run, we are going to make $b = 12$ inches. Then assume $d = 9$ ", so as to calculate the weight of the beam $12 \times 9 \times .11 = 11.88$, say 144 lbs. per foot-run, because we are assuming that a cubic inch of concrete weighs .11 lb.

$$\begin{aligned} \therefore W &= \frac{112 \times 2}{12} + 12 = \frac{368}{12} \\ &= d^2 = \frac{368 \times (120)^2}{12 \times 600 \times 12} = d = \sqrt{61.3} = 7.9 \text{ in.} \end{aligned}$$

$\therefore$ the beam would be probably made 12" x 8", and the sectional area of steel used would be $.006bd = .006 \times 12 \times 7.9 = .57$ sq. in. The tensile strength of the concrete is of course neglected. Now say we had no satisfactory way of settling the value of b , but can decide upon a suitable value of A , the conditions are not quite so simple. If f_c = the allowable compressive stress in the concrete, and D = the distance shown in fig. 416,

$$b = \frac{Wl}{5.33f_cD^2} \quad (256)$$

But W in this case equals the total load on the beam in question. On the other hand, if we want to take f_s and W (= total load) into account, we can find D by the equation

$$D = \sqrt{\frac{Wl}{5 \cdot 33 f_s b}} \quad (257)$$

Having now got all the information, including the total required sectional area of the steel, we want to know in what part of the beam the steel would be placed at its best advantage. Obviously the distance of the reinforcement from the neutral axis will depend upon the sectional area of the steel we propose to use, and *vice versa*. If this distance be called V (fig. 416), and if f_s = the allowable tensile stress in the steel, and a = sectional area of the steel we propose to use,

$$V = \frac{1}{2} \times \frac{Wl}{8 f_s a} \quad (258)$$

W being total load, and l = length of beam in inches.

Regarding the benefits of reinforcement, the conclusions of Professor Hatt, an American engineer, may be of interest to the reader, and are as follows:—

1. 1% of metal placed 1 inch from the bottom of a beam will raise the load-carrying capacity at least 3 times and the flexibility to 10 times that of a simple concrete beam.
2. 2% of metal under the same conditions will increase the load capacity 5 and the flexibility 15 times.
3. 1% of metal placed 2 in. from the bottom sets these figures at $2\frac{1}{2}$ and 12 respectively.
4. 2% of metal placed 2 in. from the bottom sets the figures at $4\frac{1}{2}$ and 18 respectively.

Now what we have just said in reference to concrete steel beams has quite neglected shear forces. In simple beams it is usually neglected. The concrete can no doubt up to a certain limit take up the shear stress of a beam when designed to simply suit certain conditions of load and span. But if we give it a little consideration we may be able to economise a little in materials. We have discussed Shear Force in previous chapters. Now when a test specimen of concrete is crushed it will fail by shear. Such shear in a beam may lead to excessive tensile stresses, and cause failure thereby. What we call simple shear may in fact be broken up into two components, one of compression and another of tension, acting in planes at right angles to each other, and both inclined at 45 degrees to the shear stress line. Say, now, we have a beam in which by the usual methods we have found the total shear on any section = Q , say I = the moment of inertia of the beam, y the distance of the point we are considering from the neutral axis, h = distance from the neutral axis to the extreme fibres. We want to find the intensity of the shear stress on any point in question, its value

$$q = \frac{Q}{2I} (h^2 - y^2) \quad (259)$$

and q is maximum when y = zero, which goes to say that the shear is greatest upon the neutral axis, and will diminish to zero at the top and bottom of the beam. Of course q will increase directly as Q . Now in a steel I beam or rectangular timber one, it is all very well to assume the material capable of standing any shear likely to come upon it. But in concrete steel beams we

have different conditions. We, in the first place, have the different materials to deal with. It is essential that these adhere firmly together; therefore it is our duty to find out if there is so much shear at the junction of the concrete and the steel as to completely overcome the adhesion between the two. Now when in any beam

- b = the breadth of that beam,
- t = total maximum breadth of steel reinforcement,
- q = value of Equation 259,
- E_1 = E for concrete,
- E_2 = E for steel,

then the intensity of horizontal shear

$$q_h = \frac{qbE_1}{E_1(b-t) + E_2t} \quad (260)$$

Then if q_h does not exceed the safe adhesion strength (= 250 lbs. per sq. inch) of concrete and steel, the beam will stand the horizontal shear set up. This shear may, however, become excessive in short and heavily loaded beams. We already discussed the principles of the design of lattice girders under chapter on Design of Structures. Let us take a beam similar to diagram 417 and assume it to be split up into bays similar to a lattice girder, a series of reinforcing bars occurring along the bottom boom. As we have to disregard any concrete below this line, simply treating it as a protective coating to the steel, the bottom of the beam is dotted.

Say now—

- w = load per lineal inch, lbs.
- N = proposed number of bays of our girder.
- h = the number of any bay in question (1, 2, 3, 4, etc.).
- θ = angle in the diagram.
- x = distance in the diagram.

Then if this is the total tensile stress set up in any of the diagonals we can find it by the equation

$$P = \frac{(N-2h)+1}{2} \times wx \times \sec \theta \quad (261)$$

and θ , which is usually 45° , we have $\sec \theta = 1.4142$. Then if we work out all these imaginary diagonals in an ordinary rectangular beam of large section, and we find a value of P greater than 50 lbs. per sq. inch (which is the limit of concrete in tension for practical calculations), we must provide diagonal reinforcing bars *rigidly* attached to the main reinforcing bars. It is upon this principle that the Kahn trussed bar has been introduced. For the same purpose stirrups are used also. Now what we have said of course bears upon beams of a uniform and solid section, and for quite moderate loads. Works are now executed by means of concrete steel in the form of large trussed girders, similar to steel ones at present in vogue. The work has, however, not received much attention at the hands of British engineers, but no doubt opens up a large field for their skill. Now we know from our theory of beams that under a uniform load we shall have the most shear at the ends and nil at the centre. The central plane of the beam, however, is purely geometrical; the result is that no part of the beam will be entirely free from shear stress in practice. This will suggest to the engineer a plate of steel

varying in thickness for the reinforcement. It might be trough or $\square$ shape. But its disadvantage is that the bond would be poor. Hence we resort to such material as expanded metal. As we stated, concrete steel beams of the lattice type have been attempted. They require, however, reinforcement in the top flange, and their calculations would be the same as in Equation 261, together with the information in the previous chapter mentioned. Unless the beam was very large, however, we would hardly alter the sectional area of the steel in each diagonal to suit the varying values of P . We should probably calculate the first diagonal and use the same section throughout, no

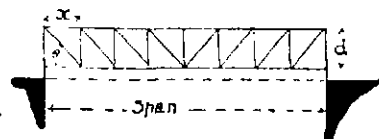


FIG. 417.

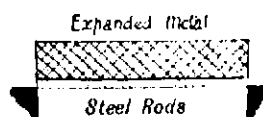


FIG. 418.

account being taken of the concrete in tension. The only time when the equation referred to would be altered is when we wish to find the compressive stress in the top boom when the struts are vertical, as in the Linville girder, in which case we have

$$P_2 = \frac{wx}{2} \times (N - 2h + 1) \quad (262)$$

w being the load per lineal inch,

x = distance between two diagonals in inches.

We shall now present to the reader typical forms of beam design in diagrammatic form. We will leave him to use his own discretion in regard to choice to suit local circumstances, and we hope that the previous reasoning will enable him to scientifically and economically design the same.

In fig. 418 we have a beam in which expanded metal is provided to take

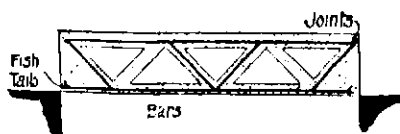


FIG. 419.



FIG. 420.

the shear. It is applied in separate sheets. The lower edges must be rapped very securely round the two outer reinforcing bars. It may, however, be formed into a trough shape. The rods are then inside the trough. If, however, the metal is of sufficient strength in itself we could imitate the rods. Figs. 419 and 420 show two types of open girders. In the trussing, ties of unequal length should if possible be avoided. The reason for this precaution in design is that any extension of the metal either from pure strain or temperature effects will be of unequal degrees in bars of unequal length, and the simpler the reinforcement the better. Some engineers have, however, in order to satisfy theoretical considerations, tried curved bars, the curve approaching the curve of bending moments. Practically, however, they appear to have found no advantage, in supported beams at any rate. Here, however, the beams become fixed, the reverse case occurs. We know

from the theory of beams that in fixed beams we have two values of BM. At the ends its value is

$$\left. \begin{array}{l} \frac{WL}{12} \cdot \cdot \cdot \cdot \cdot \cdot \\ \frac{WL}{24} \cdot \cdot \cdot \cdot \cdot \cdot \end{array} \right\} (263)$$

and at the centre

and that there are points of contraflexure at certain distances from the ends. Then between those points of contraflexure and the supports the compression occurs in the bottom of the beam, similar to a cantilever. Hence we want the metal on the top (to take the tension), changing at the points of contraflexure. But as the maximum BM at the centre is only $\frac{1}{8}$ of that in a beam merely supported, the actual depth of the concrete may be less. Hence the reason for the design of beam given in 421. Fig. 422 shows the reinforcement of such a beam securely anchored into the walls. Again we may have beams continuous over supports. Here again the stresses will change, and the reinforcement will be designed as in fig. 423, the top and bottom bars being well connected to the vertical ones. Tubular beams are doubtless quite practicable, even upon

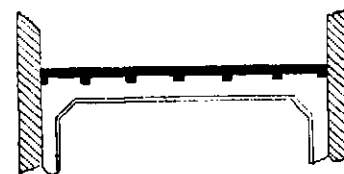


FIG. 421.



FIG. 422.

a large scale. To sum up our remarks upon concrete steel beams, we will enumerate briefly the essential points to be considered.

1. Always put the metal in its proper place, as found by theoretical results. Steel is expensive. Do not waste it!
2. All bars must have proper mechanical joints where joints are required. Avoid them if possible. Expanded metal must be securely fastened to bars.
3. Expanded metal can be recommended. It has been tried and found excellent, whether it is assisted by ordinary steel bars or not.
4. Special bars now on the market may have all the advantages claimed for them, but round bars are very satisfactory and cheaper.
5. Use thin rods, and many of them, in place of a few heavy bars of equal sectional area, within reasonable limits.
6. Do not have too much metal in one spot.

Reinforced Concrete Floors.—As continuous floors in concrete steel will receive different treatment from ordinary beams, we shall devote a few lines specially to them. Floors are in all cases divided into units by cross beams. These units are considered as Tee beams having a very wide top flange, containing rods running longitudinally and transversely, and having the usual reinforcement in the bottom, as we have already stated.

When the slab rests freely on its supports, and is reinforced in one direction only, the bending moment due to any distributed load = $\frac{WL}{8}$; if continuous over several spans, the approximate BM in centre = $\frac{WL}{10}$ to $\frac{WL}{12}$;

but it will usually be safer to assume some intermediate value, as the perfect fixing of the ends is very difficult to ensure.

For a square slab reinforced in both directions, and fixed all round, the maximum bending moment may be reduced to $\frac{WL}{20}$.

When a slab is combined with a beam, a portion of the former may be considered as forming part of the complete area of the beam; the amount to be added will depend on the proportionate dimensions, distance between beams, arrangement of loading, etc. In the case of secondary beams extending between the main beams, and used to support the floor slabbing, it will generally be sufficiently accurate for ordinary requirements to assume that the Neutral Axis occurs at the lower surface of the slab, and to take the effective width B , fig. 424, as not greater than $10t$ to $12t$, t being the thickness of the beam.

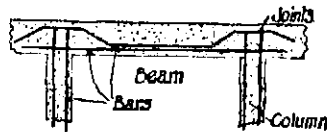


FIG. 423.

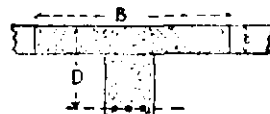


FIG. 424.

Top reinforcement may be adopted in cases where a sufficient area of concrete cannot be provided to resist the compressive stresses in a beam, and in such cases the steel bars provided may be assumed as equal to fifteen times their area of concrete placed at the point occupied by these bars. Top reinforcement is necessary when beams are carried over a support, to resist the reversed stresses due to such an arrangement.

Shearing Stresses.—In slabs where there is a comparatively large area of concrete, these stresses can usually be neglected; but in beams, and combined beams and slabs, stirrups, or the bending up of some of the tension bars, will generally be necessary. It is frequently assumed that the concrete will take up half the shear, leaving only half to be provided against by the shear bars. The shearing stress may be calculated as for an ordinary girder, and the necessary area of steel provided; this amount will depend on the system of reinforcing adopted, whether by vertical or inclined bars, etc.

Before proceeding further, let us investigate the common loads occurring in buildings in practice. We are assuming the weight of reinforced concrete to be 150 lbs. per cubic foot. This, however, assumes the best materials, broken stones or ballast. Breeze concrete will weigh about 20 lbs. less, say 130 lbs. per foot cube. Taking values of the usual superimposed loads, we have

Dwellings, tenements, flats, etc.	40 lbs. per sq. ft.
Schoolrooms	50 " "
Offices	70 " "
Stables and coach-houses	70 " "
Banks	80 " "
Theatres and churches	90 " "
Public buildings	120 " "
Drapery warehouses	130 " "
Drill-halls and dancing-halls	150 " "
Manufacturers' warehouses	250 " "

Some places, however, are apt to have a vibratory load, such as factories having much machinery and workshops. In such cases these loads should be increased by one-half, or if there is much vibration, such as a bridge for railway or tramway, or a power-house (except with turbine engines), the loads should be doubled.

The following summary of bending moments and shear forces will prove useful to the designer of concrete steel and other structural design. It assumes the beams and slabs to be freely supported at two ends.

Load.	BM Maximum.	At any Point y Units from Abutment.	Shear Maximum.	Shear at any Point y .
Evenly distributed	$\frac{wl^2}{8}$	$\frac{wl}{2}y(l-y)$	$\frac{wl}{2}$	$w(y - \frac{l}{2})$
Distributed (varies as distance from one end)	$\frac{Wl^2}{9\sqrt{3}}$	$\frac{Wy}{6l}(l^2 - y^2)$	$\frac{Wl}{3}$	$\frac{W}{6l}(l^2 - 3y^2)$
Centrally applied load	$\frac{Wl}{4}$	$\frac{Wy}{2}$	$\frac{W}{2}$	$\frac{W}{2}$
Concentrated load at point x units from right-hand support	$\frac{Wx(l-x)}{l}$	$\frac{Wy(l-x)}{l}$	$\frac{W(l-x)}{l}$	$\frac{W(l-x)}{l}$

W = total load at any point.

w = load per unit of linear measurement.

l = span in same units of linear measurement.

Then, so constructing such a series of beams having large flanges, we shall have a certain amount of intervening spaces. When these are filled up with concrete our floor is finished, the floor slabs being usually in compression, due to the load. The bars would of course be laid side by side, and continuous over the span. They will cross each other, too. The exact calculation of the actual strength would, however, be an impossibility. The above method of construction is said, however, to be stronger than an ordinary concrete filling. Although we cannot analyse the strength of such a floor, we may safely say that the slabs in question will bear twice the weight they would if merely treated as beams supported at the ends. That, of course, is a simple calculation. By means of Equations 254-5-6-7-8, then, there is a certain arching effect both anterior and due to the load. It will vary with the deflection and the design. Above 10 ft. in span we would allow 10% of the span for this effect. It is for this reason that the special arch hollow blocks in fig. 425 were designed.

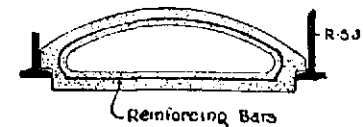


FIG. 425.

Hence, seeing our floor slabs will bear double the working load than if they were supported beams only, we, when designing floors, can calculate by any ordinary method. That is, obtain the moments of resistance, and double them, and add 10% for the arching effect.

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Now if w = weight of the concrete, obtained approximately by experiment, and w_0 = the load

$$M = \frac{(w + w_0)l}{8} \quad (264)$$

If we consider the arching effect it becomes

$$M = \frac{(w + w_0)l}{10} \quad (265)$$

Now the depth of the compression area = D .

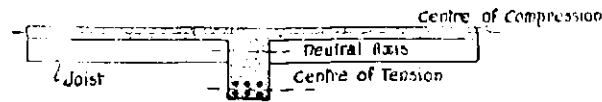


FIG. 426.

Immediately below this, of course, we have the neutral axis; then if b = proposed breadth of section and F_c the compressive strength of the concrete,

$$D = \sqrt{\frac{M}{bF_c}} \quad (266)$$

Again, if F_t = tensile strength of the steel,

D = distance from centre of reinforcement to the neutral axis, then a = sectional area of steel required,

$$a = \frac{M}{21DF_t} \quad (267)$$

D is in inches and a is in square inches.

F = lbs. per square inch and D = value found by Equation 266. As we stated before, all slabs when considered in designing are assumed to be under compressive stress, and hence our reason for designing floors as a series of Tees with enormous flanges. A typical example appears in fig. 426. The

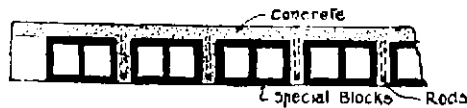


FIG. 427.

centre of compression is assumed to be in the middle of the flange.

In order, however, to use Equation 265 with safety, we must bend up the reinforcing rods at the ends and provide fishtail cuts. Shear members may also be added in accordance with our previous reasoning. An example of flooring using hollow fireclay blocks is now much in use. Reinforcing bars are also provided in the webs of the Tees. A typical example occurs in fig. 427. Two further examples of concrete floors occur in figs. 428 and 429. In accordance with our previous reasoning, concrete steel is very well applied to foundations.

Their use tends to reduce the necessary excavation. They form, in fact, when properly designed, a species of floating raft, and are particularly adapted to soft ground. The stresses set up by walls and pillars resting on such rafts cause directly opposite stresses to those in a uniformly loaded beam which is supported at the ends. This fact will be a sufficient guide to the engineer when designing. The reinforcement will take the form shown by fig. 430. Expanded metal would be useful in such cases.

One of the latest uses of Ferro-concrete is for making piles for foundations and wharf walls, etc. We have already treated timber piles and methods employed in driving them. Such piles will only give good results when entirely saturated with water at all states of tide or water-level. This consideration causes it to be necessary to start building the superstructure at a low level, a disadvantage apparent to all engineers. This is entirely obviated by the use of concrete steel piles. Their design is essentially that of loaded columns, which we are going to discuss. Another advantage is their resistance to the *Teredo navalis*. Hence their great adaptability to harbour and river work in

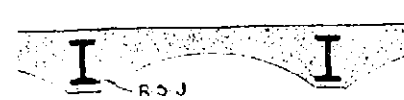


FIG. 428.

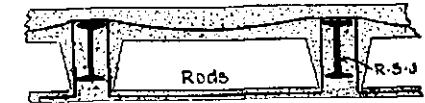


FIG. 429.

the form of wharves and jetties. Although sea water does seem to have a somewhat deleterious effect upon the concrete, yet it is not sufficient to outweigh their other advantages. They would present remarkable advantages to county surveyors who have small bridges to construct in country districts. An example of a pile upon the Hennebique system is shown in figs. 431 and 432. The usual steel shoe is of course provided, and to which the reinforcement must be rigidly attached. The special cap shown is for driving purposes.



FIG. 430.

Enlarged Sectional Plan of Pile

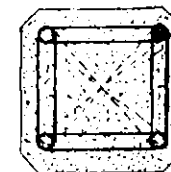


FIG. 432.

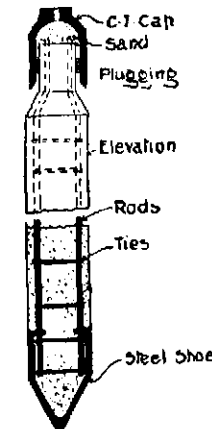


FIG. 431.

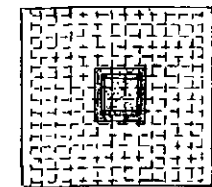


FIG. 433.



FIG. 434.

It is filled with sand and plugged with yarn to prevent the sand running out. It obviates any damage to the pile, and of course is absolutely essential to prevent damage to the pile-head and protect the vertical bars. Steel piles of a suitable cross section can also be used. Sometimes the piles are made hollow. This allows the application of a water jet to remove the sand at the toe when driving in sand. All piles must be moulded vertically, and the material very carefully rammed. The moulds must in no case be stripped within 48 hours, nor should the piles be used till 6 weeks after such stripping. The best examples of concrete piling are perhaps to be found in the town quay extension at Southampton; other examples occur at Rotherhithe. As an example of a Ferro-concrete pile foundation we have figs. 433 and 434.

wire ropes. A bottom formed of 12 in. of puddled clay should be quite as waterproof and lasting as 6 in. of concrete. The concrete should be formed

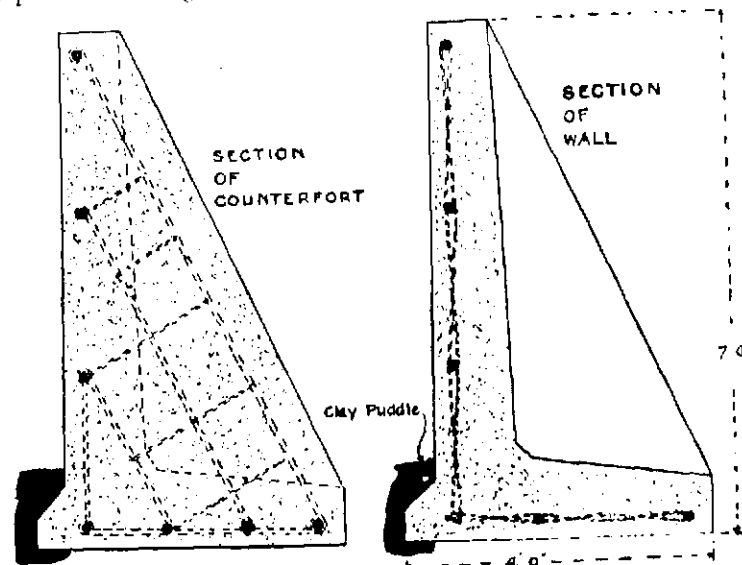


FIG. 437.

FIG. 438.

of 1 part of Portland cement, 2 parts of clean sharp sand, and 4 or 5 parts of hard broken brick varying from $\frac{1}{2}$ in. to $1\frac{1}{2}$ in.

If a central wall is put across the reservoir and the water may be at different

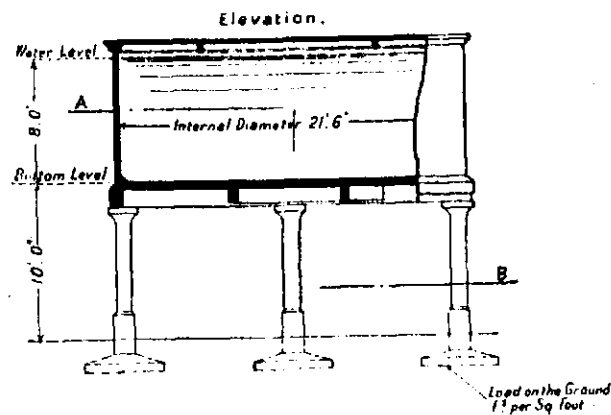


FIG. 439.

levels, the walls should be constructed as shown in fig. 437, but with a counterfort on each side and the large wire ropes through the centre of the wall. The counterforts may be placed 9 ft apart and be 9 in. thick. All the bars, ropes, and wires should be straight and taut from point to point, or

they will not take the tension properly. The concrete should be very carefully mixed, used fresh, and lightly rammed in place, so as to envelop the metal closely but not displace it.

Example III. A Reinforced Concrete Water-Tower.—This tank, illustrated in figs. 439 and 440, gives a very good example of modern Ferro concrete work. Its capacity is 18,000 gallons. The material for the drawing and the following examples were kindly supplied to me by Messrs G. Mouchel & Co., of 38 Victoria Street, Westminster. It was erected successfully at Belfast.

Example IV.—A reinforced concrete bridge erected in Ireland. The super load calculated upon was 140 lbs. per square foot. The rolling load was 18 tons. The drawing is fully dimensioned and self-explanatory, figs. 441, 442, 443.

Example V.—Figs. 444–449. A large Ferro-concrete bridge erected (1909) by Messrs Thompson & Co., Dublin, at Kilkenny, Ireland. All these drawings being of immense practical value, providing as they do illustration

Section on Line A. B.

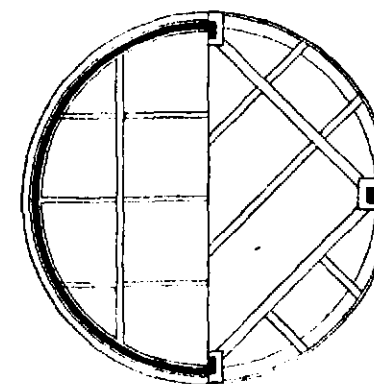


FIG. 440.

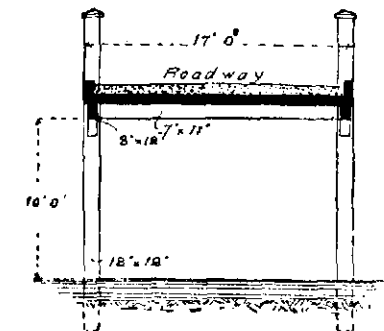


FIG. 441.

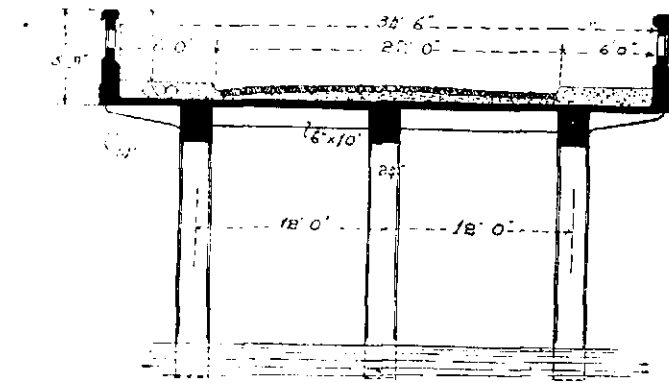
of work actually done or in progress, should be carefully studied in every detail by the reader.

Description of Bridge illustrated in figs. 444 to 449.

From the drawings we observe that the main dimensions of the new bridge are as follows:—Total length, 196 ft.; width between parapets, 30 ft.; width of roadway, 22 ft.; width of footpaths, 6 ft. each.

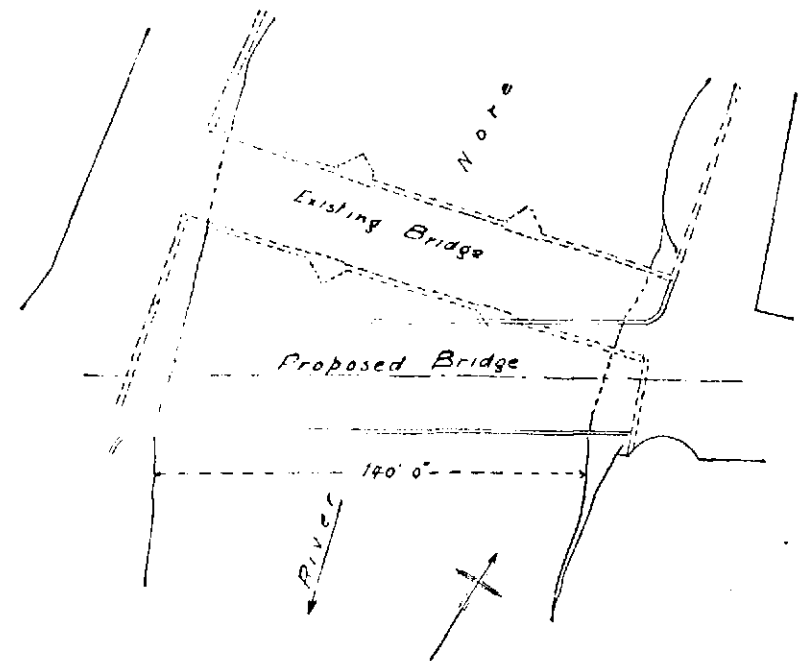
Briefly described, the bridge includes an arch with the span of 140 feet. This is the largest span in ferro-concrete of any bridge yet constructed in the United Kingdom, providing the clear headway of 11 ft. 6 in. above normal water-level, and consisting of three ribs supporting open spandrels, and connected by transverse beams and a continuous decking, the footpaths and parapets being carried on cantilever projections of the deck beams. The abutment walls and counterforts rise from ferro-concrete foundations, comprising beams and a continuous slab, built up from a series of Hennebique patent piles 14 in. square and about 35 ft. long, driven into firm strata on either side of the river, the foundation being completed by a screen of Hennebique sheet piles 25 ft. long.

The specification provides that four months after the concrete work has



- CROSS SECTION. -

FIG. 448.



- BLOCK PLAN. -

FIG. 449.

13 tons, making the weight on each beam 44 tons. The rolling loads are to be moved over the bridge as directed by the engineer, and it is stipulated that

the total deflection at the crown of the arch under the full test load shall not exceed $1\frac{1}{2}$ in., the permanent set to be not more than $\frac{1}{8}$ in. at the same point.

The new bridge, which, as before mentioned, embodies the longest span ferro-concrete arch in the United Kingdom, was designed by Mr. A. M. Burden, M.Inst.C.E., the county surveyor of Kilkenny, with the co-operation of Messrs L. G. Mouchel and partners, of Westminster.

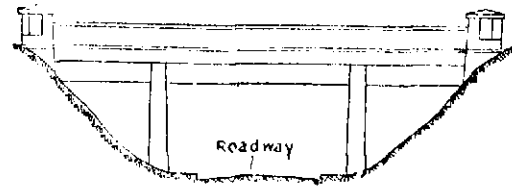


FIG. 450.

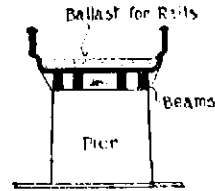


FIG. 451.

Fig. 449A gives a further example of a reinforced concrete arch bridge erected at Thorverton, Devonshire, for the County Council.

Example VI.—A bridge of different type suitable for a railway. When the girders are designed upon the principles previously set forth this will prove a most economical structure. A similar form would do well for an aqueduct, figs. 450 and 451.

Example VII.—Two examples of waterworks culverts or sewers. The

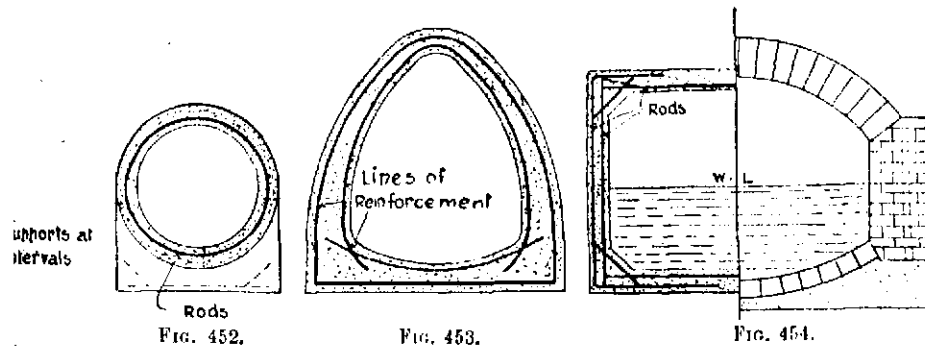


FIG. 452.

FIG. 453.

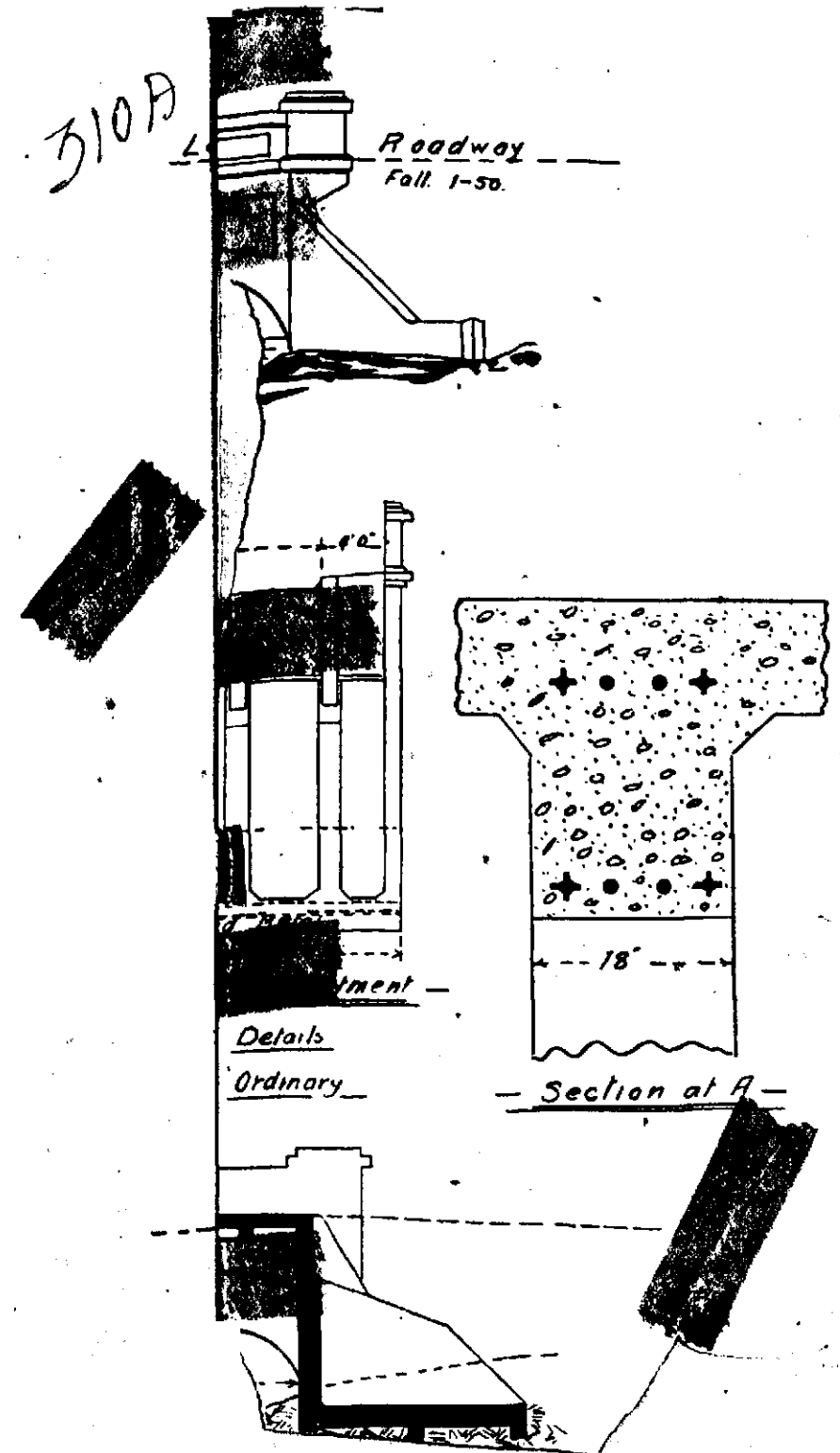
FIG. 454.

reinforcement is clearly shown. Smaller-sized sewers could be made in suitable lengths and joined together after laying, figs. 452 and 453.

Fig. 454 shows two half-sections as a ready method of comparison: the left is that of a reinforced concrete culvert, the right that of one built in brick or stone, and a glance will be sufficient to indicate the cheaper and simpler nature of the former. By means of reinforced concrete any desired dimension or shape may be adopted, independent altogether of the limits imposed when dealing with brick or stone. The smaller over-all size for any given internal area should also be noted.

The calculations necessary in designing sewers and culverts, which will be subject to an internal pressure, are briefly as follows. We will assume for the purposes of calculations any width of strip = b units.

The culverts and sewers will usually carry water, and for the purposes of



calculation we shall neglect the different densities of dirty and clean water and assume that the intensity of pressure per unit area of the sides due to a head $H = P = 2.16H$.

And to resist this internal pressure if—

S = safe stress in steel,

A = sectional area of steel required,

d = internal diameter in same units as h ,

$$\text{Then} \quad A = \frac{Pdh}{S} \quad (269a)$$

The thickness of concrete will vary according to circumstances from $1\frac{1}{2}$ " in small pipes to 6" in large sewers, as in figs. 452 and 453. This applies to circumferential bars only. Longitudinal bars are treated as beams fixed at ends, and of a span = distance between the loopings.

It may be necessary, however, to calculate upon the external pressure due to ground weight.

Find $w = \frac{1}{2}Pd_1$, where d_1 = external diameter and P = unit pressure on sides.

Let $A = lh$ = length $\times$ thickness of steel.

Let $c = 15$, F_c unit compression stress in concrete, a = sectional area of hooping.

$$\text{Then} \quad F_c[A + (c-1)a] > \frac{1}{2}Pd \quad (269b)$$

Another modern application of ferro-concrete is that of chimneys.

* "Endeavours have been made from time to time, especially in the United States, to use reinforced concrete as a material for the construction of industrial chimneys. In spite of the undoubted economical advantages to be derived from the use of this material, none of the systems so far suggested has been employed on any very extensive scale.

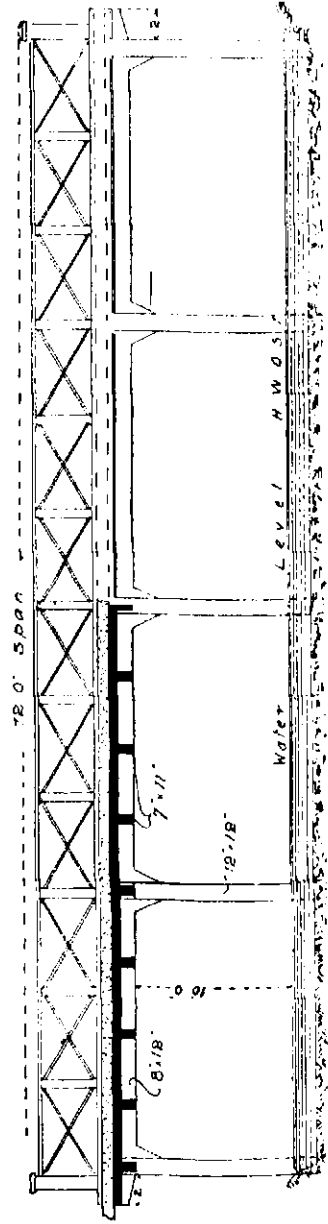
"The reason of this would seem to be that builders have so far tried to use forms and scaffolding, as employed for other kinds of concrete structures. This, combined with the expense of erecting and removing the scaffolding, more than offset the advantages of iron concrete.

"A successful attempt to build iron-concrete chimneys without any forms or external scaffolding has been made by a Belgian engineer, M. M. Dumas of Brussels. The chimneys built on this novel system (like all other concrete chimneys) are made up of a foundation, a basement, and the shaft proper. While the first two parts, which are situated close to the surface of the ground, do not show any remarkable feature, the construction of the shaft, which is composed of a number of concrete blocks 25 centimetres (9.84 in.) in height, is decidedly novel. The number of blocks obviously depends on the diameter of the chimney. Each block is provided at one of its ends with a grooved projection, so that when laid in place a continuous vertical rib is formed, into which is inserted a longitudinal armature extending throughout the height of the chimney, and being moored both in the foundation and basement. The entire chimney is thus virtually a single monolith, whose pressure on the ground is calculated in the same manner as that of any foundation.

"The blocks are moulded either in the workshops or on the site. Being reinforced with iron bars, they are well able to stand any rough handling during the transport and erection. In addition to the longitudinal armatures

* *The Illustrated Carpenter and Builder.*

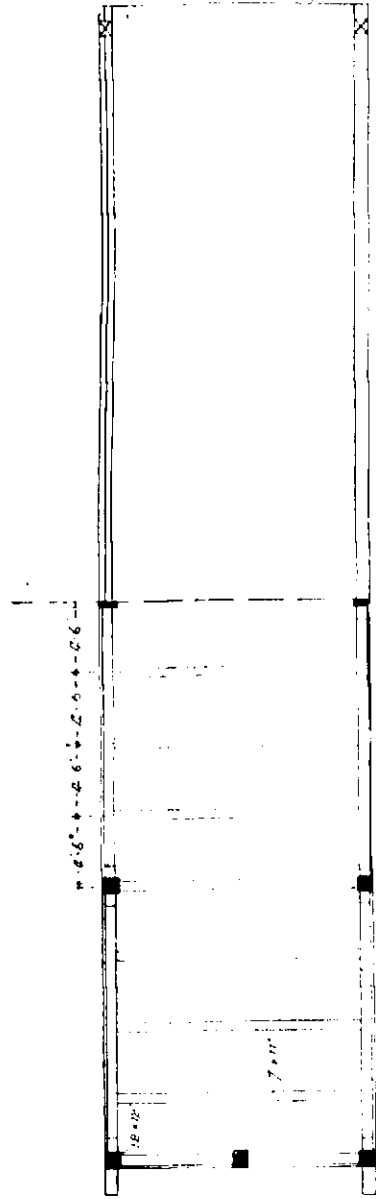
been completed and all centering removed, the bridge shall be tested by the contractor under the following loads: 300 tons of wet sand in bags



- ELEVATION -

- SECTION -

FIG. 442.



- PLAN (Looking Down) -

- PLAN (Looking Up) -

Scale of feet

FIG. 443.

distributed over the pathways and main ribs, and at the same time two traction engines weighing 18 tons each drawing two trolleys each weighing

above mentioned, the upper part of each layer of blocks is formed with a transverse armature, which takes the shape of a reinforcing ring fitted into a groove.

"The concrete blocks are mounted with remarkable rapidity: the workmen installed in the interior of the chimney set them in layers of about 12 each, after which concrete is applied on the rib 25 centimetres (9.84 in.) in height, and the joint for the next layer is completed. An important feature is that the vertical armatures are arranged outside on the chimney, so as to be always at a lower temperature than the chimney proper, which excludes any objectionable effect of a difference in expansion. In fact ferro-concrete in principle is fireproof only until the temperature of disintegration is reached; and though the coefficients of expansion of steel and concrete are practically identical, the heat conduction in the two substances is different, and, owing to the possibility of a difference in temperature between the two, the risk of disintegration would by no means be impossible.

"Another important point is the composition of the material, the various kinds of concrete being fireproof to a variable degree. The Belgian constructors accordingly used for their chimneys a special mixture of high safety. The use of a lining is recommended in the case of the presence of acid gases liable to attack the concrete or of temperature limits exceeding 800° C. (1696° Fahr.), the more so as this chimney system is especially adapted to such an arrangement.

"The chimneys terminate on the top in a cap and a cast-iron ring, which communicates with the vertical armatures reaching down to the foundation. No special lightning arresters need, therefore, be provided. As the foundation is not exposed to any heavy loads, it need not be very substantial. This entails another saving.

"Should the ferro-concrete chimney, for some reason or other—*e.g.* under the action of a cyclone or an earthquake—assume an inclined position, it would not be crushed like a chimney made of tiles, and could be straightened up at a trifling cost."

When considering this adoption of a concrete chimney-shaft, saving of space is often a great consideration.

In a shaft 300 ft. in height, for example, the brickwork at the base, according to the building regulations of our English cities and boroughs, would be about 4 ft. 10 in., whereas the thickness of the walls at the base of a reinforced concrete chimney would probably be as follows: Outer wall 9 in., inner wall 5 in., with a space of 4 in. between the two walls. Therefore considerable economy of space is attainable by the employment of reinforced concrete for chimney construction.

The shaft erected a short time ago at the works of Messrs Abraham Lyle & Sons, Ltd., of London, E., was a good example of what could be done in reinforced concrete. This chimney was 261 ft. in height and 20 ft. in diameter. The walls, for the upper two-thirds, were 8 in. in thickness, double walls being built in the lower third. The reinforcement consisted of vertical bars consisting of $1\frac{1}{4}$ in. $\times$ $1\frac{1}{4}$ in. $\times$ $\frac{3}{16}$ in. T's, and horizontal rings of $\frac{5}{8}$ -in. steel rods spaced at 18-in. centres, vertical bars extending into the concrete foundations.

The chimneys recently erected for the Cape Copper Co., Briton Ferry, Glamorgan, and for Messrs J. & S. M. Greaves, Ltd., of Belfast, were also good examples. The dimensions were as follows:—Heights 150 ft. and 200 ft.; diameters of outer shells, 13 ft. and 8 ft. 6 in. respectively.

A large chimney at Northfleet had recently been erected; it was 8 ft. 6 in. inside diameter, and 247 ft. in height above bottom of foundations, and was a fine example of reinforced concrete construction. The foundations, the bottom of which were 35 ft. below ground-level, were remarkably shallow, being only 4 ft. tapering to 2 ft. in thickness, and 18 ft. $\times$ 18 ft. in area. A 6-inch baffle wall was constructed across the lower portion of the shaft, and the thickness of the shell up to 62 ft. above the ground-level was as follows:—Outer shell 12 in., cavity 4 in., inner shell 4 in. Above this the chimney consisted of a single shell 9 in. in thickness. The reinforcement in the foundations consisted of $1\frac{1}{4}$ in. $\times$ $1\frac{1}{4}$ in. $\times$ $\frac{3}{16}$ in. T's, and in the shell of horizontal rings of steel $\frac{5}{8}$ -in. diameter, spaced 18 in. apart, and vertical bars formed of $1\frac{1}{4}$ in. $\times$ $1\frac{1}{4}$ in. $\times$ $\frac{3}{16}$ in. T's.

Although a smaller chimney, 130 ft. high, the one recently erected at the Bervans' Works, Northfleet, was of much interest. The shallow depth of the foundation was rather remarkable. While this was 15 ft. 6 in. square in area, the depth was only 3 ft., tapering to 2 ft. The inside diameter of the chimney was 5 ft. The bottom of the foundation was 16 ft. below ground-level, and this was reinforced by $1\frac{1}{4}$ in. $\times$ $1\frac{1}{4}$ in. $\times$ $\frac{3}{16}$ in. bars. For a height of 41 ft. above ground-level the outer shell was 6 in., the inner 4 in. in thickness, with a 4-in. cavity between; 9-in. diameter air inlets occurred at intervals; above this the chimney consisted of a single shell only 5 in. in thickness. The shell was reinforced by means of vertical bars and horizontal rings, the former being deflected around the smoke flues. The vertical bars consisted of $1\frac{1}{4}$ in. $\times$ $1\frac{1}{4}$ in. $\times$ $\frac{3}{16}$ in. T's, the rings being $\frac{5}{8}$ diameter steel rods placed 18 in. apart, while offset rings of 3 in. $\times$ $\frac{1}{2}$ in. flats were also provided.

The appearance of hair cracks, a frequent occurrence in a reinforced concrete chimney, need cause no undue alarm; they are rarely more than merely superficial. The vertical bars should always be turned horizontally into the foundation, while at the joints they must overlap at least 30 diameters.

Tests of Reinforced Concrete Beams.

The resistance to failure by shear and diagonal tension, and the effectiveness of metallic web reinforcement, are discussed in Bulletin No. 29, "Tests of Reinforced Concrete Beams: Resistance to Web Stresses," by Arthur N. Talbot, just issued by the Engineering Experiment Station of the University of Illinois. The bulletin, of 85 pages, gives the results of tests made in the Laboratory of Applied Mechanics of the University. The strength and quality of the concrete itself are shown to have a marked effect on web resistance, and the importance of securing a high grade of concrete in beams subject to large web stresses, even if metallic web reinforcement is used, is brought out strongly. Data are given on which to base the age necessary to enable concrete to take shearing stresses. It is established that the less the ratio of length of span to depth of beam, the greater the shearing resistance developed per unit of cross section, and also that the greater the amount of longitudinal reinforcement, the greater the shearing resistance. Both effects depend upon added stiffness, a relation which has not been formulated before. Several types of web reinforcement were investigated—bent-up longitudinal bars of different forms, unit frames, vertical stirrups of different sizes and proportions, smooth and deformed bars, etc. The notes and discussions given add to the value of the matter for those who wish to study the details of the tests. The analytical treatment of web stresses, the discussion of the

experiments, and the remarks on proportioning stirrups may all be helpful to engineers. Although a considerable fund of information bearing on the part taken by the resistance of longitudinal reinforcement and the compressive resistance of concrete has accumulated, the data of web stresses are less definite and less complete. This monograph is therefore timely, and it ought to take rank as an important contribution to the literature of reinforced concrete.

No. 511.1. At 13,000 lbs. the first noticeable crack appeared at 2 ft. to right of centre, and extended from the bottom vertically 3 in. to the level of the reinforcement, and then diagonally toward the load point 5 in. At 14,000 lbs. a vertical crack appeared 2 ft. 5 in. to left of centre, and extended to the reinforcing bars. From this point it finally ran diagonally

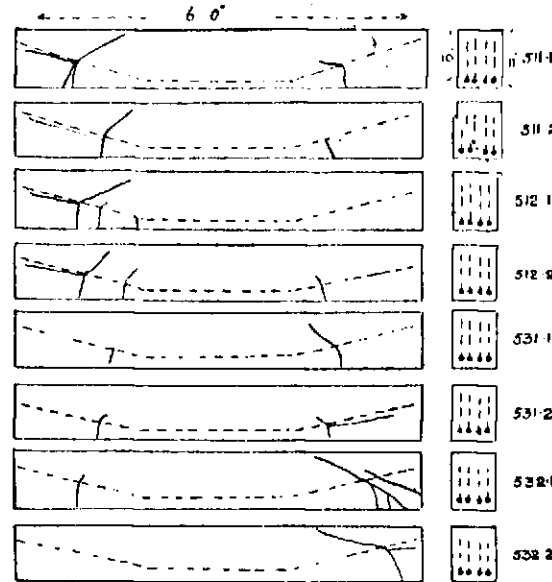


FIG. 455.

almost to the load point. The beam failed at 18,000 lbs., the crack following the reinforcing bars to the end of the beam. The load then dropped quickly to 6000 lbs., where it remained constant for some time.

No. 511.2. At 13,000 lbs. a crack appeared 1 ft. 10 in. to left of centre, and extended to the reinforcing bars. At 15,000 lbs. this crack had extended diagonally almost to the top of the beam, 4 in. to the left of the load point. The load of 15,000 lbs. was held for 15 minutes, due to a break in the machine. The beam failed at 20,000 lbs., the load then dropped slowly to 4000 lbs., and the crack on the left extended along the reinforcement almost to the end of the beam. An examination of the beam made afterward showed that the bars had slipped $\frac{1}{4}$ inch in the left end.

No. 512.1. At 15,000 lbs. two cracks were noted, one 2 ft. 5 in. to the right and the other 2 ft. 1 in. to the left of centre, extending upward to the reinforcing bars. At 18,000 lbs. a crack appeared 2 ft. 6 in. to the left of centre, and extended to the reinforcing bars, branching off diagonally to the right toward the load point and to the left along the reinforcing bars. The

maximum load was 19,000 lbs., and the load gradually dropped to 2000 lbs. An examination of the beam showed that the rods at the left end had slipped slightly. The crack had followed the reinforcing bars to the end.

No. 512.2. At 14,000 lbs. a crack appeared 1 ft. 5 in. to left of centre, and extended toward the load point. At 16,000 lbs. a crack was observed 2 ft. 2 in. to left of centre, which extended vertically to the reinforcing bars, and from here started toward the load point. The beam failed quite suddenly at a load of 17,350 lbs., and the crack followed the reinforcing bars to the end of the beam. An examination of the beam showed that the bars had slipped $\frac{1}{4}$ inch.

No. 531.1. This beam failed in a manner similar to No. 512.1, and at a load but 500 lbs. greater. The crack at 1 ft. 10 in. to right of centre appeared at 14,000 lbs., and the beam failed at 19,500 lbs., the crack following the reinforcing bars to the end of the beam.

No. 531.2. At 12,000 lbs. a crack appeared 2 ft. to left of centre, and extended vertically to the reinforcing bars. At 18,000 lbs. this crack was 5 in. above the bottom, and was extending toward the left load point. At 14,000 lbs. a similar crack appeared 2 ft. to right of centre, and extended in the same way. At 22,000 lbs. a slight hair crack was noted, following the line of the reinforcement. The beam failed by the opening of the crack at 24,000 lbs.

No. 532.1. At 18,000 lbs. a diagonal crack appeared at 2 ft. 5 in. to right of centre, and the beam finally failed along this crack. The maximum load was 20,000 lbs.

No. 532.2. At the maximum load of 18,000 lbs. a diagonal crack extended from the right end toward the load point. The bars slipped after the diagonal tension failure occurred.

The paper of course provides the results of a long series of very valuable experiments. It would be impossible to print them all in a book of this description. A few have been selected for the benefit of the reader.

For piers or structures projecting into water it will generally be found necessary to drive a series of piles, as may be required, and connect them together by means of reinforced concrete beams and bracing. The decking may be of timber, but would be better, in most cases, if of reinforced concrete flooring or slabbing. Fig. 456 indicates the manner in which a small simple pier of this nature might be built. If three or more rows of piles are used, diagonal bracing must generally be added to give increased stiffness.

A Reinforced Concrete Lighthouse.

* An interesting and novel application of reinforced concrete is afforded by the new lighthouse at One Fathom Bank in the Straits of Malacca, a

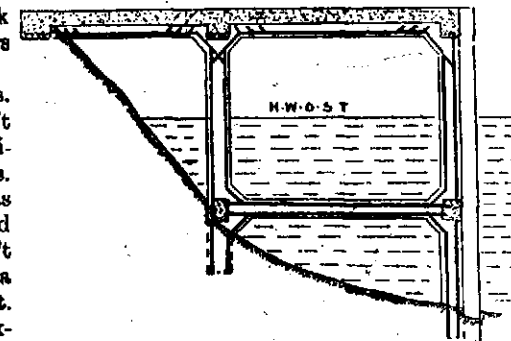


FIG. 456.

some 1-1 mortar should be used to fill up all interstices previous to the concrete being filled in.

Pivots.—The pivots should be made of the best annealed cast steel, carefully machined all over to uniform thicknesses; the rolling curves should be machined and finished to a dead smooth surface, the pivots being forced into the bearing plates by hydraulic pressure; if it be found that the faces of the pivots are not parallel with the bed-plate, then the back of the same must be remachined.

The granite bearing stones must be recessed for about 1 in. to 1½ in., and the bearing surface bush-axed or (which is preferable) sand-rubbed, as it is advisable that there should be no inequalities in the stone to prevent a perfect bearing being obtained.

The stones should be made in about 2 ft. 6 in. lengths, and the pivots should be about 1 in. less in length. The joints for these stones should be about ¼ in., and they are not to be made until after the final concrete voussoirs were filled in.

Before fixing the pivots in the stones, sheet lead about ½ in. thick should be put in the recess in the stones, and the pivots laid in and well hammered with a heavy wooden mallet; after which the stones should be fixed in position, and very great care exercised in gauging the distances between the rails so as to ascertain whether the bearing of the two rails is true; any inequality in the position of the stones could be obviated by packings laid on top of the laggings.

Use of Screw Jacks for fixing Pivots.—After all the stones are in their correct positions, it is necessary to obtain four screw jacks so as to force the pivots as close together as possible, and at the same time put a certain amount of compression on the lead seatings, when this is done, 2-in. square iron or steel bars are forced between the back of the stones and the concrete voussoirs, so as to prevent any movement when the screw jacks are taken away.

In Germany, engineers use ¾-in. bolts top and bottom, placed in the joints of the stones, but by this means very little pressure is exerted, and the bolts have to be left in until concreting is finished, and afterwards cut out, which is a most difficult and very expensive method of procedure. It is, in the author's opinion, one of the principal reasons why there is so much deflection in the arch when the centres are struck, owing to the fact that so little pressure is put on the seatings previous to concreting in.

There is no doubt that concreting exerts a large amount of pressure on the granite blocks, especially when the concrete is made fairly dry and heavy ramming is employed; but the pressure is light as compared to that exerted by four powerful screw jacks.

After all the stones are placed in position the concrete voussoirs should be filled in; the voussoirs in the centre ought to be filled in simultaneously.

Striking Centres.—It is not necessary that the whole of the outer spandril walls should be constructed before striking the centres; but if there were no urgent reason why the centering should be removed, it is better to let the same remain, as it acts as a staging to enable the masons to set the stones on the spandril faces. But within three weeks after the completion of the final concrete voussoirs it is advisable to ease the centres a trifle, so as to allow a moderate amount of pressure to be exerted on the pivots; where screw jacks are used this could be easily done without in any way setting up vibration in the arch, such as takes place when folding wedges are used.

Roadway.—After the centres are finally struck, the concrete over the arch, forming the roadway and footway, might be filled in, care being taken to make an expansion joint over the pivots in the centre of the span and also at the springing pivots. In this country a joint ½ in. is sufficient at the centre, and the same could be made of soft felt; but at the springings, if the concrete be put in in cold weather, fully ½ in. must be allowed. In forming the roadway, whether of wood paving, granite pitching, asphalt, or macadam, the expansion joint might be ignored, though one must be put in on the footways, especially if the same are made of York paving or concrete.

In order to allow the arch to expand, the outer spandril faces must not be bonded to the abutments or piers, but be recessed into the same so as to allow perfect freedom of action; this also applies to the parapet and cornice, because, if securely fixed, an accident is bound to happen in winter if the bridge is completed in summer, and *vice versa*.

When designing arches it will be necessary to refer to what we already said in our chapter upon arches. In addition we have the special calculations as follows. Divide the arch into any number of divisions = n . Consider the arch as a cantilever and find M = bending moment, due to the external loads. The crown is taken as origin, let x and y be the co-ordinates at any point positive.

H = thrust at crown

$$H = \frac{n \sum M_y - \sum M \sum y}{2[(\sum y)^2 - n \sum y^2]} \quad (269c)$$

and if M_1 = moments on right-hand side of arch,

and M_2 = " " left " "

$$\text{Let } S = \text{shear at crown} = \frac{\sum(M_1 - M_2).c}{2 \sum x^2} \quad (269d)$$

Let M_c = bending moment at crown

$$M_c = \frac{-\sum M + 2T \sum y}{2n}$$

Then depth at crown =

$$d = \sqrt{S + 1S + 0.05w + 0.025w_1} \quad (269e)$$

S = span.

w = load per foot-run.

w_1 = dead load at crown.

Compare this with previous formulæ.

Let us take the following example. We are going to add a storehouse on top of a large factory. It is to be made of reinforced concrete.

A warehouse floor is usually designed to carry a superimposed load of 2½ cwt. per foot super., for although the present use may not involve so great a load, one cannot tell what may be necessary in the future. In this case the maximum proposed load is 12 tons on one bay measuring 27 ft. 4½ in. by 10 ft., or $\frac{12 \times 20}{27 \cdot 375 \times 10} = 876$ cwt. per foot super.; but this load will most

likely be concentrated over a certain limited portion, probably bringing up the actual distribution over that portion to double the amount, say 1½

particularly dangerous point, which has been built of reinforced concrete on the Hemmehique system.

"The subaqueous portion of the undertaking proved of a difficult nature. The waves during the roughest weather often have a height of 8 ft. from trough to crest, while the rise and fall of the tide is no less than 14 ft. The base is octagonal in plan, the main building being carried on piles disposed in two annular rings—an inner and an outer—about a central pile 2 ft. in diameter, and well braced together.

"A staging was first erected on piles 50 ft. in length by 18-in. diameter, upon which was erected a temporary hut at one end to serve as a workshop and to provide accommodation for the workmen. The concrete is composed of granite metal broken up to pass through a 0.75-in. mesh in the proportion of 2.5 parts to 1.5 parts of sand and 1 part of Portland cement. The reinforcing comprised steel rods of round section and 1.75-in. diameter, laced with 0.1875-in. diameter steel wire every 6 in. For the purpose of lightness the piles were made hollow. They were made 50.5 ft. in length, eight being 18 in. square and nine 2 ft. square. They were towed over the fifteen miles of intervening water on rafts formed of twenty-six hogsheads strapped together, with the pile lying between two rows of barrels, and in order to facilitate the release of the pile at the site the raft was made in four sections. Upon arrival at the lighthouse the raft was floated under the staging to beneath the pile-driver, where a chain sling was attached to the head of the pile, the lashings with the raft cast off and the pile slowly heaved up, the raft being meanwhile gradually floated away, so as to leave the pile free when finally raised to a vertical position.

"Driving the Piles.—Heavy timber guides, clamped securely to the sides of the staging, served to keep the pile in position while it was being driven. Driving was carried out by two jets of water, actuated by a steam pump, playing at the foot of the pile. By this means it was found possible to sink the piles into the sand to an average depth of 15 ft. in from three to four hours. In the course of sinking it was found that considerable erosion of the sand bank had occurred, a result no doubt due to the obstruction offered to the course of the tides, which here run very strongly, by the piles of the staging. Under these circumstances the resident engineer decided to carry out an experiment by lengthening one of the piles by 14.5 ft., and ascertaining precisely to what depth it could be driven. The extension of the pile was carried out on the site, and it was found that hard bottom was gained after sinking the pile an additional 13 ft., or 26 ft. 8.5 in. in all. Owing to the satisfactory results that had attended the lengthening of this pile on the site, combined with the fact that driving could be carried out another 13 ft. and thereby gain a rock foundation, the other piles were similarly lengthened. The lengthening *modus operandi* adopted was to break the pile down to the hollow, which was then filled up with cement concrete of the same standard as the main fabric and a junction effected between the two parts by butting the bars of the extension in the corners of the existing pile, and covering the joint by a sleeve. Extra bars were then inserted in the sides of the pile, and laced with steel wire as before. Moulding boards were clamped to opposite sides of the pile, set plumb, and the remaining open sides then closed up with boards as the concrete was filled in and rammed down. When the moulding boards were subsequently removed it was found that the external surface of the added section was perfectly smooth.

"In carrying out this extension work upon the pile, which is centrally

placed, it was found that the concrete was somewhat porous. Since it proved impossible to keep the water out, and since the pile would have to carry a great proportion of the weight of the superstructure, it was decided to enclose it in a cylinder 5 ft. in diameter, sunk 10 ft. into the sand. When this was done, the sand within the cylinder was excavated, and the whole annular space surrounding the pile within filled with cement concrete and continued up to the level of the main bracing. The average length of the submerged portion of the piles is some 52 ft. below low-water ordinary spring tides."

Concrete Bridges.

Centerings.—The centres needed for the erection of a concrete arch are similar in every way to those required for a stone or brick arch, with this exception, that less strength is wanted, owing to the fact that a concrete arch weighs less.

Where it is required to construct the whole arch of concrete and make imitation stone voussoirs, special shuttering must be made. This could be moulded to any desired form, but great care must be exercised in fixing the shuttering and in well strutting the same, so that in making the concrete voussoirs the ramming does not cause the shuttering to bulge. The shutters for concrete voussoirs could be formed out of 3-in. to 2½-in. wood strengthened with rails, etc., the surface exposed to the concrete being machine-wrought; but it is not essential that they should be of an even thickness, as a light variation would, when the voussoir was constructed, act as a key. On the top of the laggings the thickness that the concrete voussoirs are required to be formed should be marked off, the shuttering fixed, the faces in all cases being radial to the curve, and care taken to thoroughly well strut the shuttering, so as to prevent undue distortion in consequence of the weight of concrete. After this has been done, the surface of the laggings should be covered with about 1 in. of one part sand to one part Portland cement (made dry), and then, as rapidly as possible, concreting commenced; the quicker the concrete is put in and well rammed the stronger and better it will be. When once a concrete voussoir is started it must be finished without a break; and sufficient men should be employed, so that, from start to finish, not more than two hours would elapse. When about three-fourths of the voussoir has been finished, the stone figs or iron rods, etc., should be built in for bonding the spandril walls to the extrados of the arch.

After the concrete is set, the surface should be covered over with wet sacks, and kept well wetted during the whole time that it is exposed.

Order of fixing Voussoirs.—It is advisable, when forming the concrete voussoirs, to always place the first voussoir between the points of support of the centres, not over the supports; thus, if any deflection is caused by the weight, it is put into the beam before the remaining voussoirs are made.

In closing in with the voussoirs care should be taken that the faces of the existing concrete voussoirs are thoroughly clean and well roughened; and as the concrete is brought up, neat cement grout or 1-1 cement mortar should be put on the faces of the existing concrete, so that a perfect bond is obtained between the new and the old. When ramming the concrete, care should be taken that the existing faces of the voussoirs are not in any way damaged. To prevent this it is advisable to use special hard-wood rammers. Lest, by any chance, the faces of the voussoirs previously put in should be honeycombed,

cwt. per foot super. To this must be added $\frac{3}{4}$ cwt. per foot super. for the weight of the floor itself, or $2\frac{1}{2}$ cwt. in all, making the total load to be carried by each column $\frac{27.375}{2} \times 10 \times \frac{2.5}{20} = 17.11$, say 17 tons. This will also be the load to be carried by each ferro-concrete beam between the column and the wall.

* In designing the beams the bending-moment diagram should first be drawn out as shown in fig. 456A as a guide to the reinforcement. The calculations for the reinforcement will be as follows: The floor slab has 10-ft. span, and a load of $10 \times 2.5 = 25$ cwt. distributed. The reinforcement may be assumed for trial as four $\frac{5}{8}$ -in. rods per foot-run, the sectional area being 0.9 sq. in., and the area of concrete above centre of reinforcement being $12 \times 6 = 72$ sq. in., the percentage $= \frac{0.9 \times 100}{72} = 1.25$ per cent.; then $W = (.37p + .214) \frac{bd^2}{L} = (.37 \times 1.25 + .214) \frac{12 \times 6^3}{10} = \text{say } 30 \text{ cwt.}$ Weight of slab $= 10 \times 1 \times \frac{1}{2} \times 1 = 5 \text{ cwt.}$, therefore

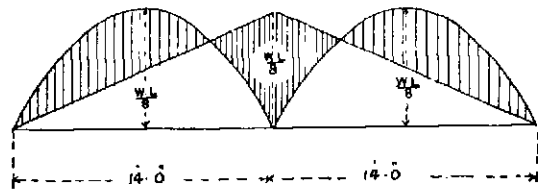


FIG. 456A.

safe distributed load $= 30 - 5 = 25 \text{ cwt.}$ The maximum bending moment on the beam between the supports is 17 ton-ft., then $\frac{WL}{8} = 17 \text{ ton-ft.}$, whence

$WL = 8 \times 17$ or $W = \frac{8 \times 17}{14} = \text{say } 10 \text{ tons equivalent distributed load.}$ The beam has a span of 14 ft. and a load of 10 tons $= 200 \text{ cwt.}$ Reinforcement $= \text{say } 1 \text{ per cent.}$, which will be given by four $\frac{5}{8}$ -in. rods. Then $W = (.37p + .214) \frac{bd^2}{L} = (.37 \times 1 + .214) \frac{11 \times 22^3}{14} = 222 \text{ cwt.}$ The weight of beam $= 11\frac{1}{2} \times \frac{11}{12} \times 14 \times 1 = 23 \text{ cwt.}$, therefore safe distributed external load $= 199 \text{ cwt.}$ The maximum bending moment over the supports is 30 ton-ft., then $\frac{WL}{8} =$

30 ton-ft., whence $WL = 8 \times 30$ or $W = \frac{8 \times 30}{14} = 17 \text{ tons equivalent distributed load.}$ The beam will be the same size, therefore the reinforcement must be increased to carry the increased load of 17 tons $= 340 \text{ cwt.}$ over the 14-ft. span, $340 = (.37p + .214) \frac{11 \times 22^3}{14}$ whence $340 = (.37p + .214)380$, or $340 = 140p + 81.32$, and $p = \frac{340 - 81.32}{140} = 1.85 \text{ per cent.}$ Area of reinforcement $= \frac{1.85 \times 242}{100} = 4.48 \text{ sq. in.}$, which will be given by six 1-in. rods. For the

* *Building World.*

column with a load of 17 tons $= 38080 \text{ lbs.}$, $W = S(A + 15a)$ or $38080 = 450(A + 15a)$, whence $A + 15a = \frac{38080}{450} = 84.6$, which would be given by a column 9 in. square with 1 per cent. reinforcement, but should not be less than 11 in. square with four $\frac{5}{8}$ -in. rods.

The general arrangement of the reinforcement will be as shown in

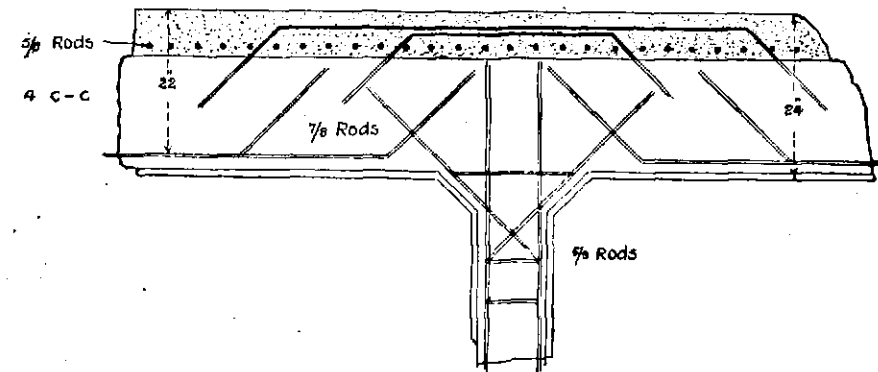


FIG. 456B.

fig. 456B, with the wall end as shown in fig. 456C, and section fig. 456D. Where the floor slab crosses the beams extra bars should be placed near the top surface to take the reverse stress, as in the case of the beams over the columns. The concrete should be 1 of cement, 2 of sand, 4 of broken stone or brick varying from $\frac{1}{4}$ -in. to 1-in. gauge."

The position of the neutral axis of a beam is a thing which is likely to give trouble to some readers, so we offer a further explanation here. Its position varies as the ratio of sectional area of concrete to steel, and this ratio should be such that the stresses both on concrete and steel are in equal proportions to their ultimate resistances, that for concrete being about 600 lbs. per sq. in., and steel 17,000 lbs. As before stated, the tension of concrete is neglected. Take fig. 456E. Here we have a triangle ABC representing the effective area resisting compression =

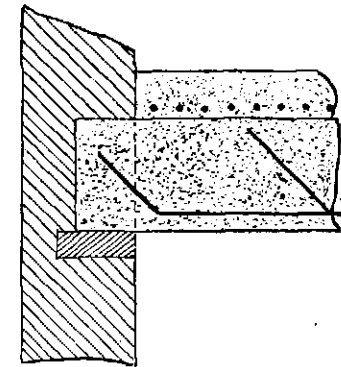


FIG. 456C.

$\frac{az}{2}$, the centre of compression being at the centre of gravity of the Δ , and

is $\frac{z}{3}$ from the top surface. The effective area resisting tension will be the net sectional area of the reinforcements. Knowing that the moments of tension and compression about any line across the section are equal, we can determine the distance z of the neutral axis from the top of the beam.

The moment of tension about a line through the centre of compression is equal to

$$ta\left(y - \frac{z}{3}\right) \quad (269f)$$

The moment of compression

$$= cx \frac{z}{2} \left(y - \frac{z}{3}\right) \quad (269g)$$

all these being equal, cancelling $y - \frac{z}{3}$ in each case. We have

$$ta = \frac{cxz}{2} \therefore z = \frac{2ta}{c \times x},$$

where c = resistance per sq. in. of concrete,
 „ t = „ „ steel,
 „ a = sectional area of reinforcements,

x , y , and z as in the diagram, fig. 456x.

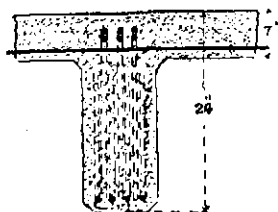


FIG. 456L.

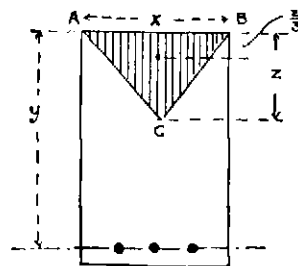


FIG. 456x.

Thus let us work out the following:—

We have a 20-in. $\times$ 15-in. beam over a 20-ft. span. Distance from top to centre of the reinforcement = 18 in. The steel is 3— $\frac{3}{4}$ in. bars (square).

Required a safe distributed load.

Bending moment (*vide* Beams) = $\frac{WL}{8}$.

Therefore equating with the internal bending moments,

$$\frac{WL}{8} ta < \left(y - \frac{z}{3}\right).$$

$$\therefore z = \frac{Rcy}{t + Rc} = \frac{15 \times 600 \times 18}{17000 + (15 \times 600)} = 6.23 \text{ in.}^*$$

* R , as used in this equation, is the ratio of elasticity of concrete to steel. In this case we assume it at 15, but 10 was suggested before as a reasonable figure; 15 may, however, be used under favourable conditions.

And as $a = 3 \text{ in.} \times \frac{3}{4} \text{ in.} \times \frac{3}{4} \text{ in.} = 1.68 \text{ sq. in.}$, we have,

$$W = \frac{8ta\left(y - \frac{z}{3}\right)}{20 \times 12} = \frac{8 \times 17000 \times 1.68 \times \left(18 - \frac{6.23}{3}\right)}{240} = 15158 \text{ lb.}$$

Actual weight of beam = $15 \times 20 \times 20 \times 150 = 6250 \text{ lbs.}$

Total load = $15158 - 6250 = 8908 = \text{say } 446 \text{ lbs. per foot-run} = \text{safe load.}$

Let us now reverse the process and design a beam for an evenly distributed load of 12,000 lbs. = say 500 lbs. per foot-run over a span of 24 feet.

Take $y = \frac{1}{12}$ span = 24 in.

$$\text{Then } z = \frac{Rcy}{t + Rc} = \frac{15 \times 600 \times 24}{17000 + (15 \times 600)} = 8.3 \text{ in.}$$

Now $BM = \frac{WL}{8}$, but to get this accurately we must have the actual weight of the beam.

Let us assume it equal in breadth to

$$x = \frac{y + z}{2} = \frac{24 + 8.3}{2} = 13 \text{ in.}$$

Approx. weight of beam = $\frac{26 \times 13 \times 24 \times 12 \times 150}{1728} = 8450 \text{ lbs.}$

Total load = $12,000 + 8450 = 20,450 \text{ lbs.} \therefore$ proceeding in the same way as our previous example,

$$BM = \frac{WL}{8} = ta\left(y - \frac{z}{3}\right) \therefore a = \frac{WL}{8t\left(y - \frac{z}{3}\right)} = \frac{20450 \times 24 \times 12}{8 \times 17000 \times \left(24 - \frac{8.3}{3}\right)} = 2.0 \text{ sq. in.}$$

Say we use 3— $\frac{7}{8}$ bars = total sec. area = 2.297 sq. in.

We determined z experimentally.

Let us proceed and check our assumptions.

$$\text{Now } ta = \frac{xcz}{2} \therefore x = \frac{2at}{cz} = \frac{2 \times 17000 \times 2.0}{600 \times 8.3} = 13.65 \text{ in.}$$

The next thing to investigate is the intensity of shear stress.

Shear = $\frac{W}{2}$ in an evenly loaded beam. Now the effective shear-resisting area of the concrete

$$\begin{aligned} & x\left(y - \frac{z}{3}\right) \\ & \frac{W}{2} = \frac{20450}{2} = 10225. \\ & \therefore \text{shear stress per sq. in.} = \frac{10225}{x\left(y - \frac{z}{3}\right)} \\ & = \frac{10225}{13.65\left(24 - \frac{8.3}{3}\right)} = 35.2 \text{ lbs.} = S. \end{aligned}$$

We have a factor of safety then of about 2. The adhesion of the concrete and steel should also be worked out by Equation 245, but when the shear is in safe limits the adhesion will usually be so too.

Let us consider floor slabs.

We have a factor to use

$$= \frac{L^4}{L^4 + B^4} = f.$$

B and L = length and breadth of slab = say 10 ft. and 8 ft.

Then BM

$$\frac{WL}{8} \times f$$

= taking W = total load at 23,000 lbs.

$$= \frac{23000 \times 8 \times 12}{8} \times \frac{10^4}{10^4 + 8^4}$$

$$= 276000 \times .7 = 193200 \text{ in. lbs.}$$

$$\therefore f a \left(y - \frac{z}{3} \right) = \text{moment of tension.}$$

$$\therefore z = \frac{Rcy}{t + Rc} = \frac{15 \times 600 \times 4}{17000 + (15 \times 600)} = 1.38 \text{ in.}$$

Equating to find a we have

$$a = \frac{BM}{t \left(y - \frac{z}{3} \right)} = \frac{193200}{17000 \left(4 - \frac{1.38}{3} \right)} = \frac{193200}{60180} = 3.20.$$

$$\therefore \frac{3.20}{10} = .320 \text{ sq. in. of reinforcement per ft. length of slab,}$$

Say we use 3 - 1/3 in. square bars per ft.-run.

Reinforced Concrete for Retaining-Walls.*

Dealing with retaining-walls, it is stated that reinforcement is estimated to effect a saving of between 70 and 80 per cent. of concrete, besides saving space and time. The footings may be in the form of a solid slab, upon which the earth presses, so giving stability. The upright wall is rigidly attached to this slab, and may be quite straight, have a slight batter, or lean backwards. Where the wall is not of great height, say not above 16 ft. to 18 ft., this form answers perfectly, the upper portion of the wall being designed as a cantilever; but where the wall is taller, or the outward pressure great, it is advisable to throw backward wings or buttresses from the top of the wall to the inner end of the foundation slab; then the wall acts as part of a beam between the buttresses. Where the wall is not of extreme height, and it is desirable to use the square at the back of the wall, the toe may be greatly prolonged, dispensing with the foundation at the back. In such a case the wall should have a slightly increased batter, and receive extra reinforcement at the base angle.

* See also chapter on Walls for practical example.

The wall is composed of shallow alcoves, strengthened at the back by a series of buttresses, spaced from 10 ft. to 15 ft. apart. Their thickness and length must be determined by the forces they have to resist, their elevation, and also the nature of the ground. The walls between the buttresses are, as we have said, curved inwards. They may be perfectly vertical, and about 1 ft. thick if not more than 15 ft. high, but if taller, or the ground is of peculiarly treacherous nature, it may be well to give the walls a slight cant backwards from base to top, and to step them, giving them greater thickness at the bottom and narrowing upwards. But the walls should be of equal thickness in the horizontal direction. The ends of these walls should be continued into and bonded with the buttresses, which should project slightly, this joint partaking somewhat of the nature of a dovetail. In this way the outward thrust is transferred entirely to the buttresses. But in order to resist lateral pressure and the forward thrust, reinforcement is necessary, and this takes the form of square or round iron rods, placed from four to eight in a row horizontally, bedded in a thin layer of concrete. The ends of these rods are provided with eyes, through which a transverse bar is threaded and brought as near to the head of the buttress as practicable. These beds of concrete, with their rows of rods, are placed every 5 ft. or so, the distance apart, the number and size of rods, and the depth to which they are carried into the buttress depending upon the stresses and the character of the ground.

If such a wall is used to support a roadway it will be necessary to mask its indented form, and this may be done by sealing thereto a slab of armoured concrete, which may be utilised as a footpath with an added parapet. On the other hand, if the wall is placed in a tree-planted avenue or public garden, the alighting can be made deeper at little extra expense, and utilised as recesses for semicircular benches.

CHAPTER XXI.

MASONRY BRIDGES, WEIRS, AND RIVER WORKS.

IN previous chapters much has been said bearing on the subject-matter of this chapter. For instance, arches, brickwork and masonry, centering foundations, retaining-walls and roads all belong to the subject of Bridges, and it is by means of a combination of the principles set out in these respective chapters that the engineer will be able to successfully design a masonry [with which we include Brickwork and ordinary Concrete (not reinforced)] bridge of moderate span.

The main object in the first part of the present chapter is to consider the all-important matter of the site of the proposed works. Its selection must in all cases be in the hands of engineers of experience, as each case requires special forethought. In the second part of the chapter we shall consider weirs on rivers for water control, because the two works seem to be somewhat closely allied to each other, inasmuch that when a new bridge is constructed, that bridge may be a suitable site for a weir to control the hitherto free and ungoverned stream or river, and serves as a suitable platform on which may be placed the gear for controlling the sluices and gates.

When choosing a site, if intermediate piers on the river are going to be adopted, sharply curved parts of the river must be judiciously avoided, and the reason is that the consequent rapidity of the current which occurs on the concave bank of such a bend will tend to undermine the foundation. Skew bridges are not to be advised unless special circumstances warrant their adoption.

The piers must be carefully designed so as not to obstruct the clear waterway, and to gain this essential object they must be exactly parallel with the current, and have pointed or cylindrical cutwaters at each end. They should not be thicker than is consistent with safety.

The spring of the arches must be above the highest ordinary water-level, and flood-level too, if possible. In all cases, however, sufficient waterway must be provided for the greatest known flood, and the form of arch which gives the greatest free waterway for a given rise is known as the Hydrostatic arch (*vide* Arches).

In any case the crown of the arch should be at least 3 ft. above flood-level, to allow of the passing of floating bodies. Now very often it will occur that the land for some distance on each side of the river will be flat, and particularly liable to floods. These lands usually act as channels for flood discharges. If that is the case it is folly to attempt to bring the proposed line of communication to the river bank on an embankment; a viaduct the whole way across to well out of the reach of such flood waters must be provided. On the other

hand, if such lands only temporarily fill up with more or less stagnant water, the bank may be formed having culverts in its length of brick or iron to discharge the superabundance of such waters. It is usual to provide a gradient up to the centre of the arch from each side. This, which should be within the limits of the road construction (or railway, as the case may be), is to provide the greatest possible headroom for floods with the greatest economy of material. The next thing to consider is an unyielding foundation for the piers and abutments. When, however, there is any likelihood of the piers settling, the arches must be made specially stable. It is the practice to construct such piers of absolutely solid masonry. It is considered by some engineers, with whose views the writer concurs, that this is generally a bad practice, more especially where the ground beneath such piers is of a yielding nature.

The duty of a pier is merely to support the arch and transmit its weight to the foundation. Thus a solid pier would be practically equal to half the weight of the arch which it supports. Of course it would appear that to reduce the weight of the actual pier by reducing its thickness was a bad practice, and so it is. We have a required sectional area to provide in order to support a superincumbent load, but a thickness of $\frac{1}{4}$ of the span is quite little enough; hence if that condition provides an excess of masonry, it will at once appear to the engineer that to reduce the weight we will have to provide arched openings or recesses in the pier itself, and provide also inverted arches at the bottom of such openings likewise. But such openings should not be more than $\frac{1}{4}$ of the superficial area of the whole pier.

The same reasoning applies equally to abutments. They may be then designed in the form of wall in fig. 285. Now in all such piers and abutments, unless there is a rock foundation, settlement will naturally result. They must be well built up to the springing courses, and loaded with a load in excess of that which they are going to carry when the arch is finished. In such a state they should be left for some months, and water admitted to the interior of the cofferdams. This is to ensure that the piers will be truly under *natural* condition. Then before such loading is applied the engineer will provide a firm bench mark, and take very accurate levels on to it and the piers. These he will note down. He can then test the settlement at intervals; and when it appears to have *entirely* ceased, the arches may be commenced.

The voussoirs of the actual arch must be cut with the greatest accuracy by experienced masons.* The joints should be as thin as possible so as to avoid much after-settlement, which should not in a properly constructed arch be more than 3 in. on a 100-ft. span. The centres would be constructed as pointed out in chapter on Constructional Carpentry, and must be very *stiff* in form, so as the load of the arch stones will not tend to alter their shape. If this is not duly considered, the weight of the arches will cause the centre to spring up in the middle. This will necessitate their being loaded to bring them back into position.

On the Design of Small Road Bridges in general.

A county surveyor usually has under his charge a number of small road bridges, say up to 30 ft. span. Their object is mostly to overcome small obstacles, such as ditches and streams, and they generally take the form shown in figs. 463-4-5. A great many of such works were executed in the early

* See chapter on Masonry and Brickwork.

part of the last century when steel was not used for the purpose, and Portland cement unknown. Hence we usually find them built of brick or stone, set in ordinary lime mortar. Many of these bridges are now in a fine state of preservation, but it is usual to find many of the approaches steep and tortuous, and with the arches springing from below the river-bed. The crowns usually raise objectionable humps on the road surface. They are usually too high, that is, the height of the soffit of the arch gives no assistance to the passage of flood waters, while the spans are insufficient for this purpose. Again, in spite of the fact that such bridges usually find place on swift mountain streams, no provision is made to prevent scour by means of any paving. The bridges are usually insufficiently strong to carry traction engines and steam rollers.

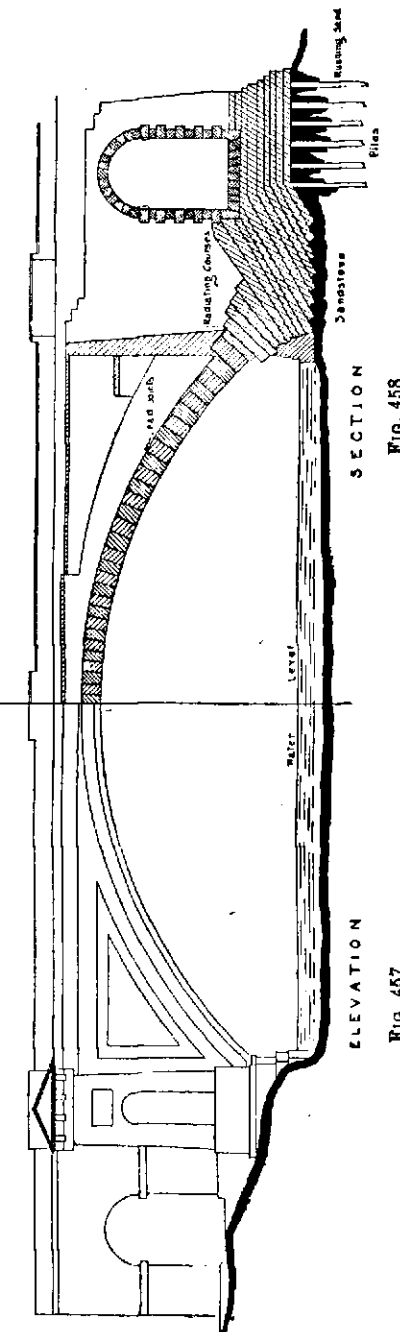
The first consideration is design, that is, from an architectural point of view, while the clear span and height under the floor are matters of local consideration, and also as to the number of spans. Small bridges have rarely more than one span, and in any case piers have the disadvantage of impeding the flow. Hence we conclude that a flat-topped bridge is most suitable for roads having straight approaches on an easy gradient, and on most roads 10 ft. is ample for the width of roadway. This, however, will not allow of two vehicles passing on the bridge, which, on a main road with much fast motor traffic, is a disadvantage, and the width should be the full width of the roadway, together with one or two footpaths, according to circumstances. Over small mountain streams it is an advantage for the arch to spring from the river-banks, so as not to interfere with the flow, and the discharging capacity must be at least equal to the bridge above it, if there is one.

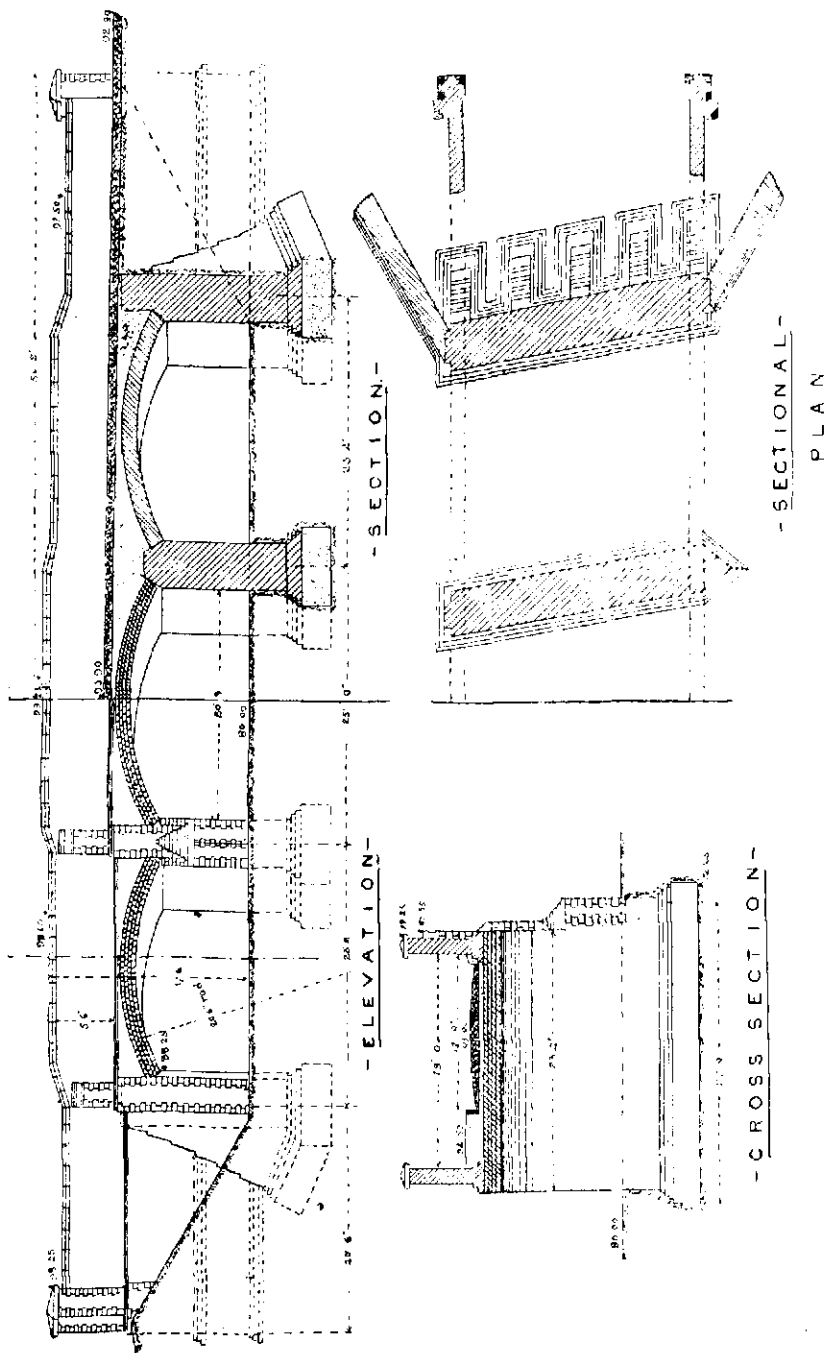
As stated, a skew bridge is to be avoided. At the same time it is a bad practice to place a bridge at right angles to a tortuous stream and the road, which more than likely runs parallel with the stream, turned abruptly to meet it. This is often unavoidable in crossing railways. In such cases, then, unless it would be policy to straighten the course of the river for some distance, a skew bridge will often prove the best, because any interference with the natural course of a river must be undertaken with caution, by reason of the fact that it will always tend to resume its former direction. The most important consideration of all is the foundations, the nature of which must be ascertained before any attempt can be made at the design. It is not usual to find gravel in this direction, and the foundations should be carried down at least 5 ft. under the lowest level of the river-bed, unless hard rock is encountered. In the meantime concrete bags are very useful where water is troublesome, while on bog land (much of which is encountered in Ireland) a raft foundation of timber (old railway sleepers) planks laid at right angles to the face of the abutments are very useful, especially in such cases where there is about 18 in. of hard earth above the bog, which it is unwise to disturb. Trial pits are of great service in ascertaining the exact nature of the subsoil, but the money available for road bridges is usually not enough to warrant their use.

The piers and abutments of small bridges are not usually subject to much calculation, 3 to 4 ft. being the usual mean thickness, the face being vertical and the back stepped as pointed out under Retaining-Walls. If for arched bridges, they must withstand the greatest thrust of the arch; and if for supporting girders, the greatest dead and live loads on the bridge, and also the thrust of the earth backing, due not alone to its actual weight, but to the horizontal component of the live load on the roadway immediately above it.

Concrete abutments are economical, especially if *displacers* are used.

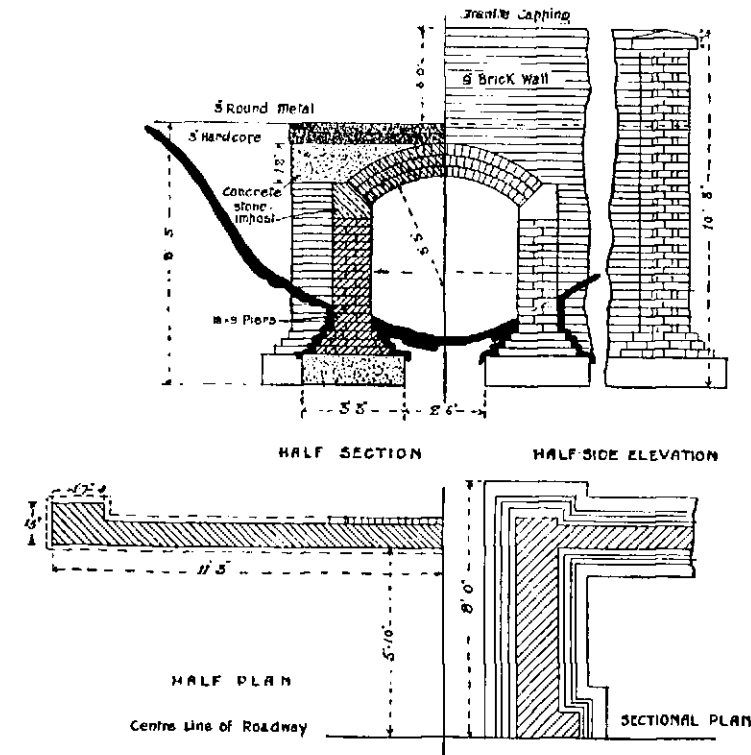
These should be of rough stone, and carefully set to break joint, and should be at least 6 in. apart on plan. They should be laid flat, and not greater in size than 18 in. x 12 in. x 9 in. The faces of concrete piers and abutments should, of course, always be finished in 1 to 1 cement. Piers, if high, sometimes have battering faces. The walls which are built up from the arch to roadway level are termed *spandril walls*. They must be of sufficient strength to withstand the pressure of the filling and the horizontal component of the load on it in the same way as the abutments. It is also usual to introduce two or more walls between the two outer spandril walls and parallel to them. They are known as *internal spandril walls*, and they are generally covered with flagstones, and the space below left hollow, or small arches may be turned between them instead, to support the road metalling. When it is necessary to build these spandril walls of any height, they are strengthened by cross walls parallel to the abutments (see fig. 458). And it must be remembered that where the arched construction is employed, the outer walls are subject to the outward thrust of these arches. Immediately above the keystone of the arch a string course is run along the entire length of the outer spandrils, and they are continued upwards above this to about 3 ft. 6 in. above roadway level. These walls are known as the *parapet walls*, and they are usually 12 in. to 18 in. thick, and surmounted by a skew or half-round coping, the latter being preferred in concrete work. In the case of heavy banks to support the approaches, wing walls must be built to retain them. They are subject to the same calculations as retaining-walls, but usually have a sloping face. Squared rubble masonry, laid dry and pointed in lime mortar, is a favourite and inexpensive form of construction. The back next to the road they retain is usually vertical. In most cases it





The figures 97-20 are heights above Ordnance datum.
Figs. 459-462.

will be advisable to pave the space between the abutments, and the best construction for the purpose is that of heavy stones 18 in. deep laid on edge, with the longest dimension up and down stream. They should be well bonded together, and form, when complete, a channel of circular cross section. Care must be taken, however, that the ground below is not likely to be undermined by the current, and a stone apron on the downstream side is an advantage in such cases, to reduce the velocity of the water between the abutments. Concrete paving, though easy to construct, is not to be advised, owing to its more or less rapid wear under such conditions.



Figs. 463-465.

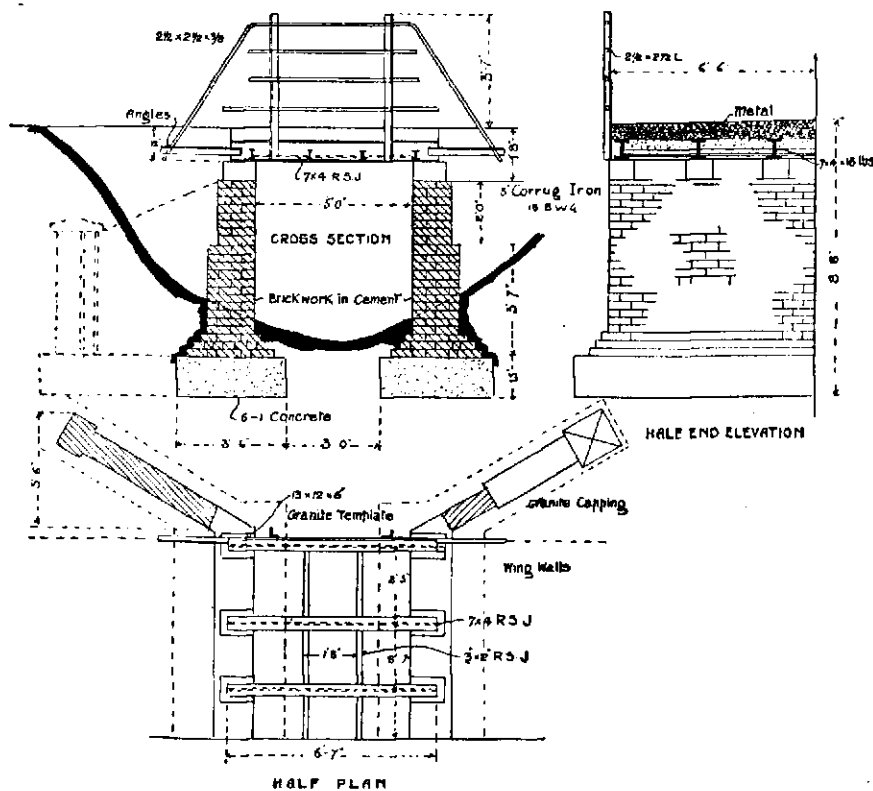
In our chapter on Centering we mentioned the Grosvenor Bridge at Chester as being a good example of a masonry bridge, as well as the centre upon which it was constructed. We will now place before the reader the details of the bridge in question. Fig. 457 shows the elevation and fig. 458 a half section.

The piles on the back portion of the bridge abutment were necessary because an outcrop of loose sand was here encountered, the rest of the foundations being upon solid rock. The bridge is made of sandstone, of which a good quality was found near the site.

The faces of the abutments, however, and the first courses of the arch are of granite. The quoins are of limestone. A special feature of the bridge is

the radiating courses of the abutments, and the steps cut in the rock on which they lie. Now a special precaution was here adopted to prevent the joints opening, provided in the shape of the lead sheets on the joints up to about one-third of the arch, because at this point it is found that the centre of pressure passes very near the centre of the arch stones (*vile* Arches). The result was a very even settlement. This has also been adopted with much success by some continental engineers.

Another example of a masonry bridge occurs in figs. 459, 460, 461, and 462.



Figs. 466-468.

It is fully dimensioned. The main structure was built of brickwork in cement, the quoins and copestones of granite, the arches of Stafford blue bricks of a special hard quality, set in cement. The foundations are of concrete upon compact vegetable earth. The figures thus—(52.3)—refer to Ordnance datum.

Figs. 463-465 give examples of a small road bridge for a country district, all instances of what might any day confront the county surveyor. The example is a good one, and the drawings show full details and dimensions. In such cases there will often be little chance for the engineer's discretion as to whether it would be advisable to construct his bridge of masonry or steelwork, seeing steel sections are so cheap nowadays. It may be that this would

be the most economical course. Be it so or not, we have given in figs. 466-468 the same conditions fulfilled by a steel superstructure.

In figs. 469-470-1-2-3-4 we give examples of a timber and steel footbridge for a 50-ft. span. It is 3 ft. wide, and is constructed across a river. The floor of the footbridge is economically constructed of old railway sleepers. In the case in question there happened to be some old piers in the river which were solid enough for the particular purpose. They were 3 ft. wide and 16 ft. apart. The main girders were built up of steel sections (BSB 9) $6 \times 4 \frac{1}{2} \times 20$ lb. If more suitable (BSB 10) $6 \times 5 \times 25$ lb. could be equally well used, as shown in elevation in fig. 469, and section in 470. They

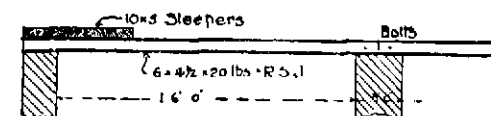
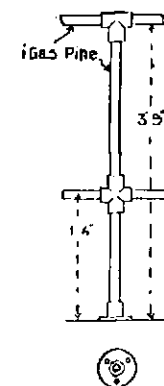


FIG. 469.



FIG. 470.

are connected by 1-in. bolts having double nuts at each end. The handrail is formed economically of gas pipes having flanges, crosses, and tees as shown in figs. 471 and 472. The old sleepers must be cut so as to bear upon the girders quite level, and are held down to the girders by dogs (fig. 474). They may also be fixed together by dogs as shown in fig. 473. We shall, of



Figs. 471 and 472.



FIG. 473.

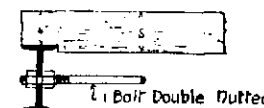


FIG. 474.

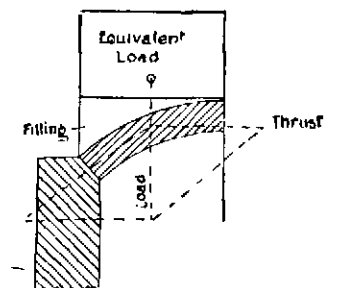


FIG. 475.

course, devote a special chapter to Bridges of Steel; and it may be suggested that we have departed from our reasoning by the last two examples; but being of so small a nature, we considered they would be out of place in the chapter mentioned. Regarding the actual stresses set up in an arch under load, we usually consider the heaviest load likely to be encountered will be that of traction engines and steam rollers, and we design our bridge accordingly. In addition to our previous reasoning under Arches, we have the following graphical methods of stress determination.

First set the actual bridge in diagrammatic form to scale upon paper. Take the external load at say 5 cwt. per sq. ft., and add the extra weight as shown in fig. 475, considering it at 1 cwt. per cub. ft. Then find the centre of gravity of the whole, and draw a vertical line through this point to

The abutments of bridges which withstand the thrust of an arch will be designed as retaining-walls; and knowing the direction of the resultant thrusts and the magnitude, the matter should not present any real difficulties. We have, however, a formula which will be of assistance in the design.

Let $S = \frac{1}{2}$ span,

„ $V =$ versed line (for explanation *vide* Chapter VI.),

„ $T =$ thickness of abutment at the springing.

All units are to be taken in feet.

Then

$$T = 17 \left(\frac{S^2}{V} + V \right). \quad (271a)$$

This applies to flat arches having a span greater than 150 ft.

For semicircular arches T may be taken as the thickness of abutment at a height above the springing $= \frac{2}{3}$ the radius.

As before stated, piers are usually made $\frac{1}{2}$ span of arch in thickness, but in so doing, especially in high piers, it must be seen that the safe intensity of stress is not exceeded. For rubble stone 8 tons per sq. ft. is ample, and 3 tons for good concrete; soft sandstone would not be safe with more than $2\frac{1}{4}$. On rock foundations, however, we can safely have a load of 9 tons per sq. ft., and on ordinary earth $1\frac{1}{2}$ –2. Foundations in sand should be at least 3 ft. deep and in clay 4 ft. It is very important for the engineer to rigidly inspect the site of the piers when the founds are opened up. If the ground is not of a uniform character, unequal settlement will more than likely result. Broken rock, mud, soft clay, and running sand are always treacherous. All bad places should be cleaned out and filled up to a level surface with masonry or concrete. Undermining of piers must be specially guarded against; induced scour will increase this tendency, and very thick piers will cause induced scour. Therefore in bad ground it is prudent to increase the sectional area by adding length to the piers where such a procedure would be deemed advisable. The chief difficulty in bridge work is the cofferdams for putting in foundations under water; in soft ground piled foundations have to be used. An example of such a foundation, together with a masonry pitching between the piers, is shown in fig. 476b.

River Works.

We now pass to the consideration of the other part of our chapter: river works, and the form of weirs and training-walls, etc.

Regarding the hydraulic properties of rivers, and the measurement of the flow of water therein, we shall treat the same under the heading of Hydraulics. It is well known that all river channels on their way to the sea increase in width at a more or less uniform rate, and the cross section of the bed usually consists of a concave curve, and the course of a river is sure to be more or less devious. All rivers are liable to silt up in places, to the general detriment of navigation and the régime of the stream in general.

But in beds of hard rock there is usually a scouring tendency, which, however, usually will occur in the upper reaches. It is these actions of deposit and scour which engage the attention of the engineer who wishes to improve a river channel, and he can so direct his work that these actions are hastened or retarded as circumstances will require. By the laws of hydraulics, if it were not for friction the velocity of a river would continually increase. But

the opposite effect usually takes place. In the lower reaches we have a part of the river where the velocity falls off rapidly, and we have more velocity in mid stream than at the side, which tends to keep the bed clear. This is the navigable channel, and is called the "*thalweg*." Now, before commencing improvement works on a river, we have to carefully find out, usually from previous records, what the usual variations of volume are. Circumstances, of course, alter cases, but in British rivers the lowest levels usually occur from June to September, and the floods from December to March. Now, contrary to what hydraulic laws might lead us to believe, we find practically in all cases that the width is greater than the depth. This proves to us that the bottom of a river offers more resistance than the banks, and also the natural effect of gravity upon the detritus. The reason why river channels are tortuous is because one bank (the concave one) is of a softer material than the other.

The two main objects of river work are—

1. Improvement of the navigable channel.

2. The prevention of floods.

It is the second to which an engineer in municipal practice usually is required to turn his attention. Other works of a more or less subsidiary nature are—

3. Water for power purposes.

4. „ irrigation „ „

5. „ domestic supply.

6. „ canal feeding.

Regarding navigation, it usually requires the uniform deepening and straightening of the thalweg and providing a towing-path. These paths must be above ordinary flood-level, and are usually placed on the side of the prevalent winds, that is, the bank of the river nearest the west. The width is usually 12–13 ft.

Suppose, however, we have a river which suddenly widens and gets shallow. Our navigation will suffer. This has been successfully rectified by planting aquatic plants on the sides of the wide parts. These will rapidly grow and collect the silt, and train the river into the central channel, with a consequent scouring effect. If, however, the river is subject to heavy floods, the practice becomes useless. The plants soon get removed again. We then have to resort to groins carried out from the banks. These check the flow of the river, and catch the silt and deposit it, and they act, in fact, in the same way as the groins for raising the foreshore where sea erosion takes place. A good form of groin for both sea and river use is illustrated in figs. 479 and 480.

Now we mentioned the word régime in connection with river works. A river is said to be in a state of régime when the materials of the bed are able to resist a tendency of the current to sweep them forward. Channels in a rocky bed will usually be in such a state, but in earthy beds it is either *just* in a state of régime or not so at all; the result is a series of transverse ridges with very gentle slope, a sort of wavy, undulating shape, due to the rolling motion of the detritus. When a river is straight it will soon become curved, and the curved parts, due partly to centrifugal action, will tend to become more curved till the tendency is resisted by a natural obstacle in the form of a rock boulder.

One of the most important engineering works in connection with rivers is that of preventing erosion of their banks, and to give them additional stability to prevent such wear.

represent the load. Then draw the parallelogram of forces as shown at right angles to the crown, and impost and complete it. Scale off the results. We can then see if the load, which will be the load per foot-run of the arch, is too

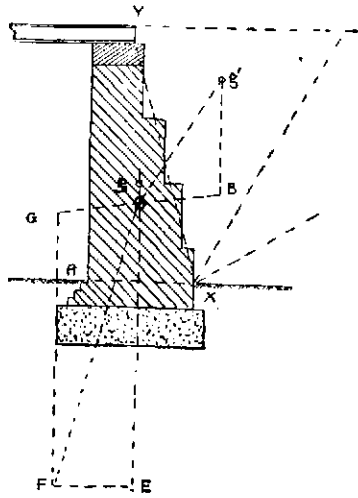


FIG. 476.

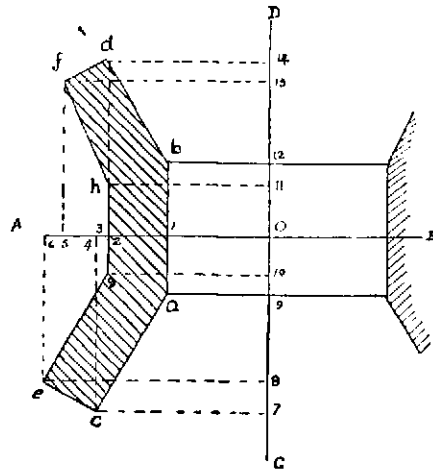


FIG. 476A.

much for the material of which the arch is built. If not, all well and good; but in any case the abutment must be capable of withstanding the thrust, and must be well backed with earth, rammed in layers. Another problem

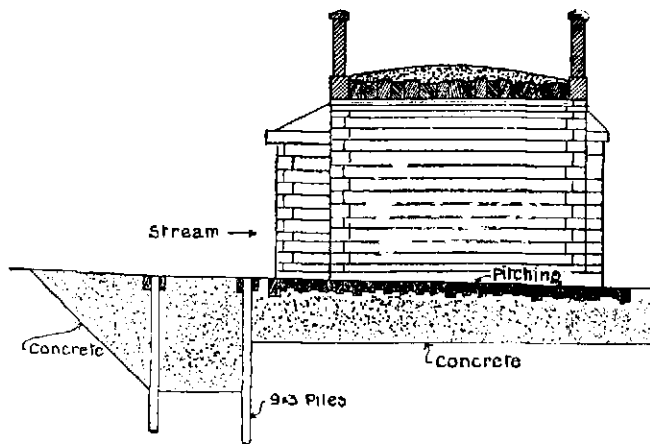


FIG. 476B.

which confronts the bridge designer is that of the abutments. These are in many cases masonry retaining-walls, but their treatment is somewhat different to what we have before described. Take fig. 476. First set out the ordinary ground-level as near as possible, and at the opposite side of the wall, on the ground-line,

draw the natural angle of slope, and in order to get the wedge form for our calculations draw the line XY. Find the centre of gravity of this wedge = g' in the usual manner, and drop $g'B$ perpendicular to any convenient scale = to the weight of the earth wedge and any superincumbent load which it may have, either dead or live, and divide this by the distance between the wing walls. Now draw $g'D$ parallel to the line of rupture, cutting $g'E$ in D. BD will then equal the total thrust on 1 ft. of the abutment. Now in the section of the abutment above AX find g = the centre of gravity of the masonry and draw a vertical line to meet GB in D. Set DE = weight of the wall between the wing walls + half the dead and live load upon the bridge, and divide again by the width between the wing walls. Make FG = DE and complete the parallelogram, and we have DF as the resultant, which of course should be in the middle third of the masonry (this applies to girder bridges only). Fig. 477 shows how a curved wing

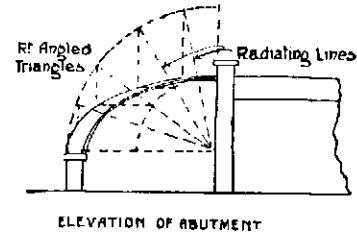


FIG. 477.



FIG. 478.

wall may be set out. First draw a correct plan of it, and then draw the elevation of the two piers. Then draw any two quadrants (dotted lines); draw the radiating lines at intervals as shown, and upon them construct little right-angled triangles. These will then give the required profile as shown. We gave before an equation for the point in an arch to which the solid backing must be carried. For convenience we will give another one, see fig. 478. Then if r = radius of the arch at the crown and t = thickness of the arch, the distance

$$x = rt \quad (270)$$

to find the level of the point of rupture = y ,

$$y = \frac{tr + .096r^2}{2t} \quad (271)$$

The following is a practical method of setting out the abutments of bridges. Obtain a tracing of the foundations, preferably to a large scale. Set up a theodolite at the point O, and set AB and CD at right angles to each other, and set out points on AB and CD so as pegs may be driven in and a cord stretched between them. The co-ordinates must be drawn on the tracing as shown by the dotted lines fig. 476A. Now get two tape lines and tie them together, hold their ends at the points f and c , etc., and by setting off the distances as read on the tracing we can set out any required point, and drive pegs in. They would then be connected all the way round by a cord, and the ground rutted along this line to mark the excavation. For foundations of skew bridges it is only necessary to give AB and CD the required skew.*

* From *Surveying*, by T. Whitelaw, jun.

As in the case of training the thalweg, the simplest and cheapest way to effect this object is the planting of the banks with aquatic plants. Where these, however, would inconveniently or dangerously impede the flow, we have to resort to other methods. These usually take the form of (1) groins, (2)

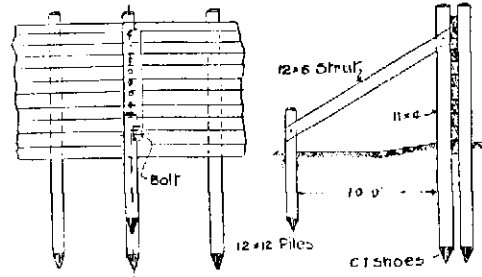


FIG. 479.

FIG. 480.

retaining-walls, (3) pitching of stone, (4) cribwork, (5) timber sheeting, (6) fascines. Regarding the groins, we have already described them, but, in addition to the wooden form, which perhaps are more suited to sea defences than to river work, we may have sort of dykes of rough stones or wattled stakes. The distances between such groins would be about five times the length of the groin.

They are really only more or less of a temporary improvement.

Dry-stone pitching is much used to protect earthen banks on rivers, especially the wash of passing steamers where navigable; this also applies to ship canals. The slope should not be steeper than 1-1. The thickness,

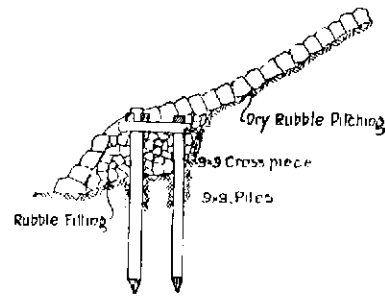


FIG. 481.



FIG. 482.

which would be about 1 ft. at the top, would increase downwards at the rate 1 in. per foot. There must be also a suitable foundation, either of piles, as in figs. 481 and 482, or bags of gravel or cement. These piles with their attendant walls and connections would be subject to the usual calculation for strength. The pressure which would come upon them would be found by the formula—

$$\frac{PR}{L} \dots \dots \dots (272)$$

P = weight of pitching.

R = rise of slope, L = length of slope.

Cribwork is used where timber is both abundant and cheap. It is much favoured by American engineers. It simply consists of layers of logs pinned together to form a frame. This is floated to the site of works, loaded with stones, and sunk.

Timber sheeting would essentially consist of a form of sheet piling of the usual form, securely anchored to the bank by means either of concrete blocks

or plates of iron. Iron or steel in rods would be used. These plates would be nearly vertical and subject to the following calculations. Say—
H₁ and H₂ = depth of the top and bottom edges below the surface respectively.
W = weight of 1 cub. ft. of the earth.
θ = its angle of repose.

S = the holding power of the plates.

$$S = W \times \frac{H_1^2 - H_2^2}{2} \times \frac{4 \sin \theta}{\cos^2 \theta} \dots \dots \dots (273)$$

Then in the plate we have a certain point to which the tie rod must be attached. This point is the centre of pressure, and its position on the plate is below the surface of the ground = 1)

$$= D = \frac{H_1^3 - H_2^3}{H_1^2 - H_2^2} \dots \dots \dots (274)$$

Reinforced concrete slabs would be of great value in such work.

A good example of a timber facing is given in fig. 483. Guide piles are driven, either vertically or slantwise, connected on top by wales, lined inside with planks and backed with earth. It is a costly procedure. All river works want a deal of forethought and experience before being put into action. For one thing, the river should not be trained very far out of its natural course, especially on rivers having a sandy bed, because the resulting currents will probably overthrow the new works. Straightening rivers should only be practised to a limited extent. Nature must, in fact, not be opposed, but made to correct itself. An example of river regulation by means of longitudinal and cross dykes is given in fig. 484.

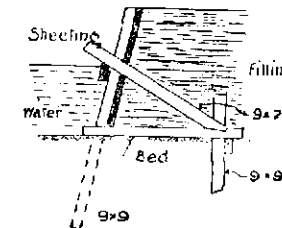


FIG. 483.

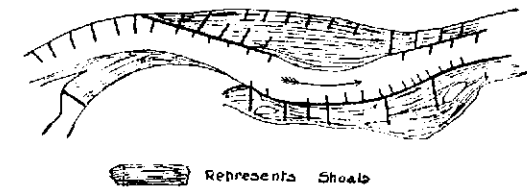


FIG. 484.—Use of Dykes in River Works.

Weirs are embankments or dams placed across a river channel to raise the bed, and probably to form a store reservoir. They are more often than not erected for power purposes (*vide* Water Power), sometimes for navigation.

The first thing when considering a weir is to think which is the best position for it. As we stated in regard to river bridges, the site of a weir must not be on a curved part of the river, lest the concave bank should be undermined. They are very often curved in plan, and sometimes V-shaped, the V pointing upstream. In any case the crest should be slightly lower in the centre than at the sides, say an inch or so, according to circumstances. The upstream side of a weir would be steep, having either a vertical face or a slope of about 1-1, not more.

The top, usually 2-3 feet broad, is level, but the front is flatter, 4-1 or 5-1 being usual slopes. This is to avoid a rush of water and cause it to strike the bottom obliquely.

Faces, in the form of a series of steps, have been tried too, but they are mostly applied to the waste weirs of reservoirs. By forming the weir, of the form shown by fig. 485, in a series of ponds, of not more than 6 ft. of difference of level between them, the descent of the water in the heaviest floods can be

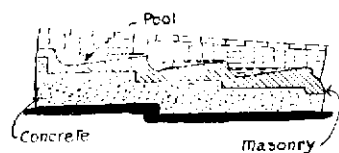


Fig. 485.

easily checked and gradually let down. It is also usual in ordinary weirs to give the front slope a short projection at the base horizontally, and slightly curve it up at the end. That the ends of the weir tail into firm ground at the banks is most essential. A weir may be made of dry stone having a steep slope on the upstream, and a flatter one downstream. But it will be necessary to have one or two rows of timber piling to keep the stones (which on the face should be roughly squared and set in courses) in place. Such a weir is illustrated in fig. 486.

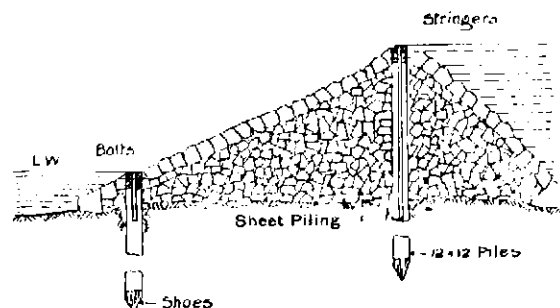


Fig. 486.

The plan could either be as in fig. 487 or 488, in each case making the total length of the weir three times that of the river breadth. This, however, is a somewhat large margin; at the same time ample length should be provided to

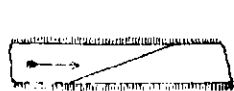


Fig. 487.



Fig. 488.

prevent the water rising to too great a height on the crest. Now a solid masonry or concrete weir would be built in the same way upon a firm foundation as any other masonry structure; and where the foundation is soft, piles have to be driven in the usual manner. It is here the judicious use of reinforced concrete could be adopted with merit.

If, however, the foundation be of timber piles, a row of sheet piles at the base on the upstream side would be necessary to prevent the pressure forcing its way into the foundation. The masonry is usually set in cement mortar, and in the heart may be rubble. But the facing of all weirs, whether of masonry or concrete, should always be of dressed ashlar (see Masonry), or block in course, the stones on the crest being doweled into each other, or

metal cramps used. They should all be laid as headers, too. A form of masonry weir based on the foregoing principles occurs in figs. 489.

Now in all weirs of any size a means has to be provided of letting off rapidly, and often at very short notice, the flood waters, by means of sluice gates. These are of various forms, but usually consist of rectangular or circular cast iron gates in culverts, or sometimes of timber. The method of raising them varies with circumstances. Small ones would be worked with a rack-and-pinion motion or screw. In all cases a counterpoise weight is

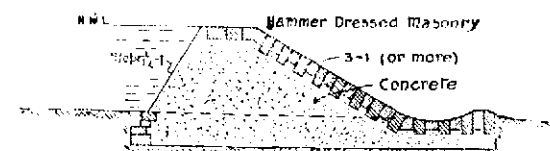


Fig. 489.

provided to allow of the gate being raised with the least possible labour, because under heavy pressure the friction of the gate against the slides is considerable. This condition practically limits the width of any one sluice to 4 ft., with a total sectional area thereon of 16-20 ft. If the culvert is then insufficient to deal with the entire flood water they must be increased in number. Some sluices are raised by hydraulic power, if such be handy, especially those in connection with large canal locks, the pressure acting directly upon a ram or piston, which in turn acts directly, without the intervention of gearing or other appendages, upon the sluice. Figs. 490 and 491

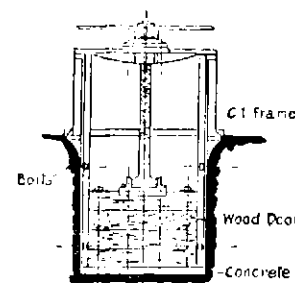


Fig. 490.

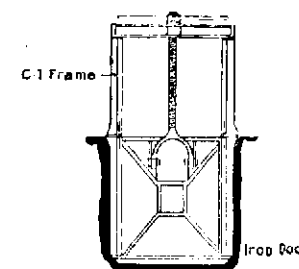


Fig. 491.

show small sluices worked by manual labour, while 492 and 493 show submerged sluices for dock gates worked by hydraulic power. Perhaps the most successful sluice yet made is that invented and patented by Mr Stoney, and manufactured by Messrs Ransomes & Rapier of Ipswich.

An example of a Stoney sluice occurs in figs. 494-5-6-7.

This plan comprised a county road bridge 30 ft. wide between parapets, with 4 segmental arches of 24 ft. span and 5 ft. rise, springing at 9½ ft. over the floor, which consisted of a solid bed of Portland cement concrete 2 ft. thick, terminating up and down stream in masonry curtains, whose foundations, as well as those of the abutments and piers, were laid 5 ft. below floor-level.

Four sluices, specially designed by Mr F. G. M. Stoney, were to be placed 4½ ft. above the upstream face of the bridge, the piers and abutments of which were extended for that purpose. Recesses were also left in the

prolongations of the piers and abutments, in which were to be fixed the cast-iron jambs and the vertical roller paths of the sluices, so that the waterway of the bridge would be in no way obstructed when the sluice gates were raised.

The sluice gates each measured 25 ft. long and 8 ft. 9 in. high. They had a skin of boiler plate $\frac{1}{4}$ in. thick, riveted to vertical rolled-iron joists, which were supported by two main beams placed horizontally at equal distances above and below the centre of pressure. These two beams, each formed of two channel irons, were trussed, and their ends were connected and carried two vertical frames of cast iron, whose downstream faces were planed, and composed the movable roller paths. Roller frames were interposed between these planed faces and the vertical rails or roller paths fixed in the recesses of the piers.

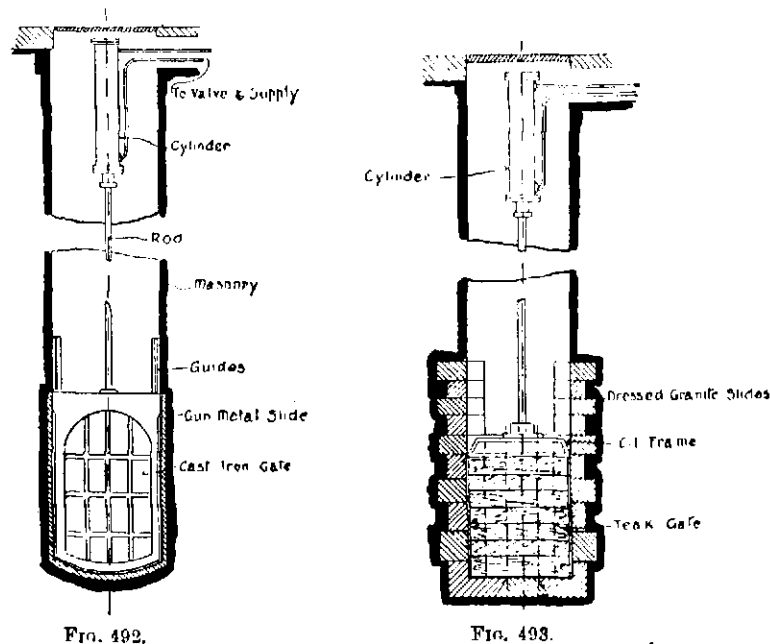


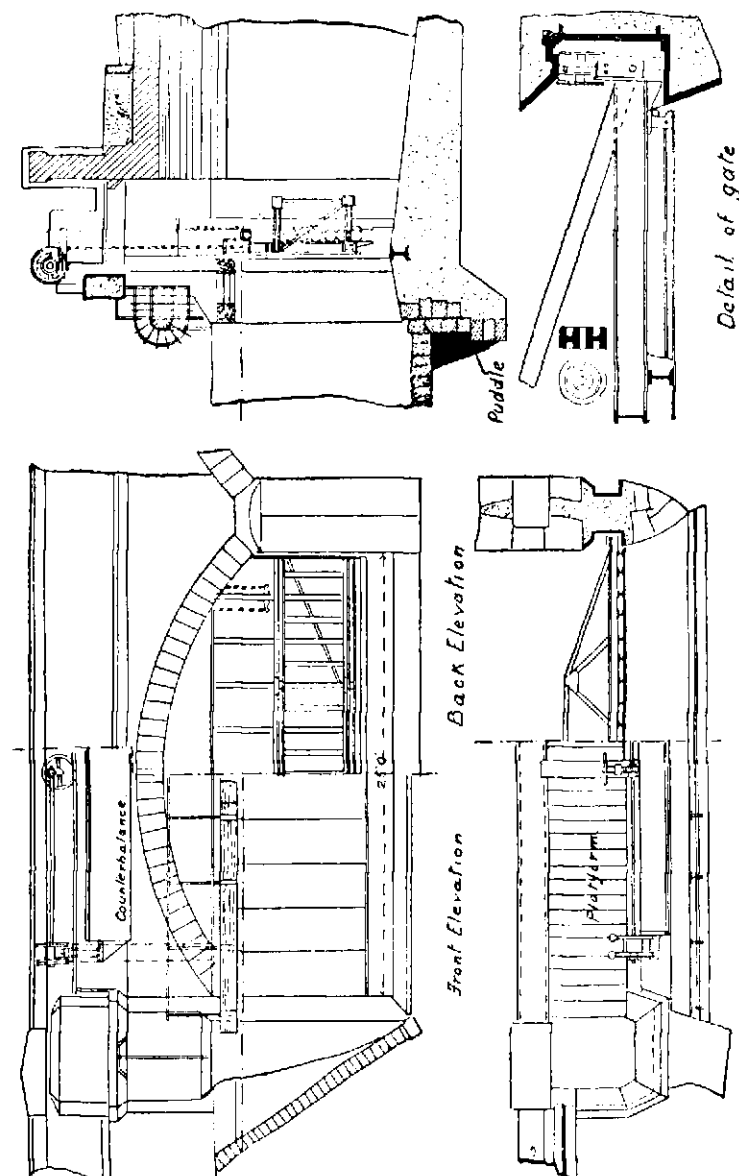
FIG. 492.

FIG. 493.

The roller frames each consisted of a pair of steel flitch plates 6 in. wide, kept apart by and securing the ends of the axles of 7 cast-iron friction rollers $8\frac{1}{2}$ in. diameter. These rollers were spaced in the frames 8 in. apart from centre to centre, and their faces were carefully turned to true cylinders, so that they should bear evenly on the planed faces of the roller paths, which were 4 in. wide.

The roller frame was suspended by a pulley in the bight of a small chain. One end of this chain hung from a bracket fixed on the top of the roller path, and the other end from the top of a small pillar surmounting the movable roller path fixed to the beams of the gate. This ensures the roller always being in its proper position in case the gate was out of water and would be swayed by the wind. The sluice gates when shut down rest upon a cast-iron frame embedded in concrete and laid truly level upon the upper

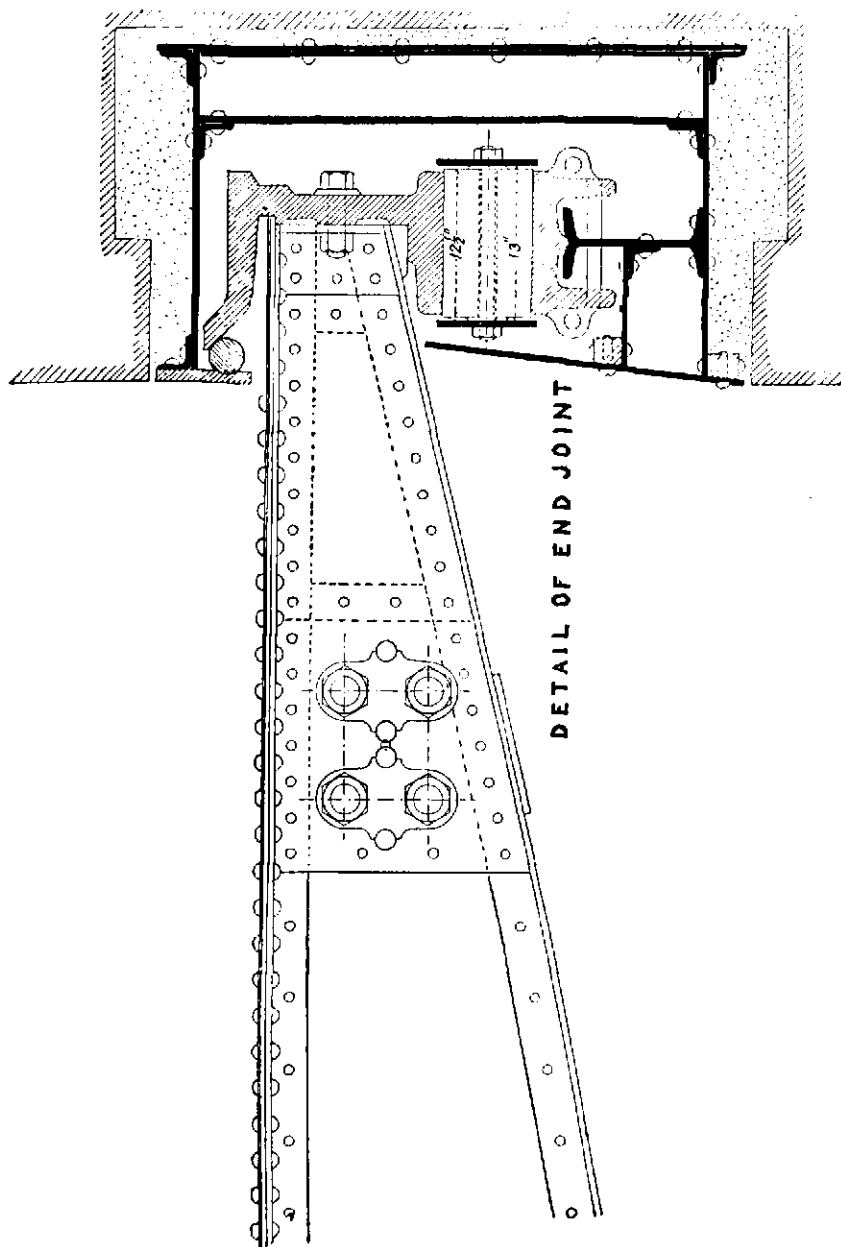
surface. The lower edges of the gates are also faced in a planing machine. By use of rods of iron suspended from the top corners of the gate, and kept in



Plan

FIGS. 494-497 See also Figs. 497A and 497B.

contact with the ends of the gate and the flanges of the jambs by the natural water pressure, not a drop of water will pass through the space (about $\frac{1}{4}$ of an inch) left for clearance when raising or lowering the gates.



the question of deposition, the oblique weir, fig. 487, was designed to remedy the evil, but its advantages are questionable.

Besides the types of sluices already described, there is a form called the

draw-door weir. It consists of a row of sluices sliding vertically in grooves in the piers. They are a useful adjunct to the overfall weir for letting off flood waters. Coming now to more elaborate constructions, we have the needle weir, shown diagrammatically in fig. 497c, consisting of a series of spars which rest against a horizontal bar at the top and a masonry sill at the base. The needles (or spars) are lifted out by hand; sometimes a small winch is

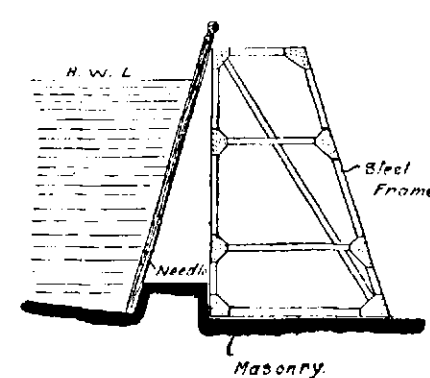


FIG. 497C.

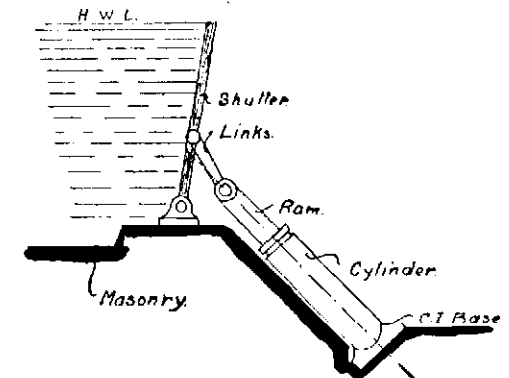


FIG. 497D.

provided for the purpose. A permanent or temporary platform would run along the top of the steel frame. Another system is that whereby a steel frame is placed across a river and has a series of shutters, made something after the style of flexible shutters over shop windows. This is rolled up by machinery on the framework, and placed on a trolley and moved away till

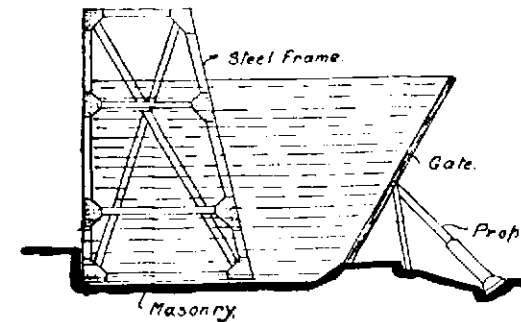


FIG. 497E.

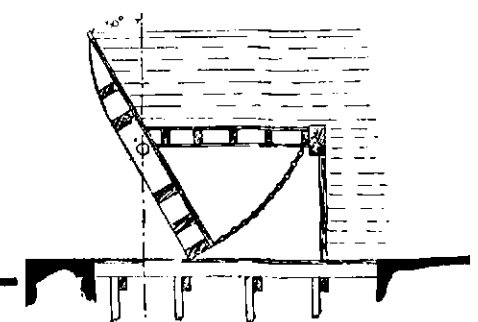


FIG. 497F.

again required in dry seasons. It would take too much space to describe them here, but to those who wish to pursue the subject of river engineering further Mr Vernon Harcourt's *Treatise on River Engineering* will prove of great value and interest. Again, there is the shutter weir, which normally has a horizontal position, and so allows free passage of the water. It is raised by means of a hydraulic ram or other motive power from the horizontal position through a system of levers, as in fig. 497D, or a system of chains, as in fig. 497E, a system of props falling into position automatically, which could be released at will.

Whenever the volume in the river is in excess of all requirements the surplus is passed under the gates, and it was found quite a simple matter to so regulate the height of the gates that the surface of the water upstream should be maintained at the proper level. Each gate is perfectly independent of the others.

The sluices are counterbalanced by troughs of iron filled with gravel and cement. The chains which connect each gate with its counterpoise pass in two pairs over large grooved pulleys of 20-in. diameter. The four pulleys are all keyed on to a shaft extending over and along the girder which supports the pulleys and the service platform.

This shaft is bushed where it passes through each pair of pulleys, and the annular space between this bushing and the hollow cylindrical bearing barrel is filled with a live ring of free rollers.

The bearing barrels are balanced on knife-edged supports, which are riveted to the top flange of the girder. This ensures an equal strain on each of the pair of suspending chains, which were 12 inches apart. The knife-edged bearings of each pair of pulleys were 20 feet apart, so that the weight of the sluices and counterpoises is borne at a distance of 2 feet from the bearings of the girders, which had a clear space of 24 feet between the corbelled wall plates.

The weight of each gate was $4\frac{1}{2}$ tons, and of each counterbalance also $4\frac{1}{2}$ tons.

The greatest pressure to which these gates could be subjected was when the water in dry summer weather fell below the level of the sill downstream, or, in other words, when the back of the gate was dry and the water upstream was just tipping its crest.

The hydrostatic pressure due to this head of 8 ft. 9 in. was a little over 26½ tons, and the friction due to this enormous pressure (which would put ordinary gates of the same size entirely out of court) is almost entirely eliminated by the free rollers, which are, in fact, the essence of Mr Stoney's invention.

One of the best examples of river control is to be seen on the Manchester Ship Canal at the lock gates. The sluices, which are Stoney's, are similar to the type just described, and were erected by Messrs Ransomes & Rapier, and have always given satisfaction.

Figs. 497A and 497B show details of modern Stoney sluices to a larger scale than the figs. 494-497.

Fig. 497A shows a section through the centre of a sluice, together with the raising gear, platform, and roadway.

In fig. 497B we have a sectional plan of the end joint, showing the rollers and vertical rods referred to. The steelwork is all very clearly shown, the drawings are of the actual sluices now in operation on the Manchester Ship Canal at Latchford Docks, Warrington, where they have always done excellent service; and I am much indebted to Mr W. Henry Hunter, M.Inst.C.E., Chief Engineer of the Manchester Ship Canal Co., who kindly supplied me with the two drawings.

We proceed to investigate a few novel varieties of sluices, which, though not often seen in England, are a type which have found favour amongst continental engineers. The overfall weir which we have been describing does no doubt effect its object of pounding up water, but will tend to cause deposit in the river channel by reason of the consequent reduced rate of flow. This especially applies to lower reaches and to large rivers in general. It may

here be stated that, like water power, river engineering is not a branch of the profession which demands much attention in England, because of the practical unimportance of our rivers for inland navigation, and the decay which such navigation has been of late years subject to, due no doubt to the paralytic effect

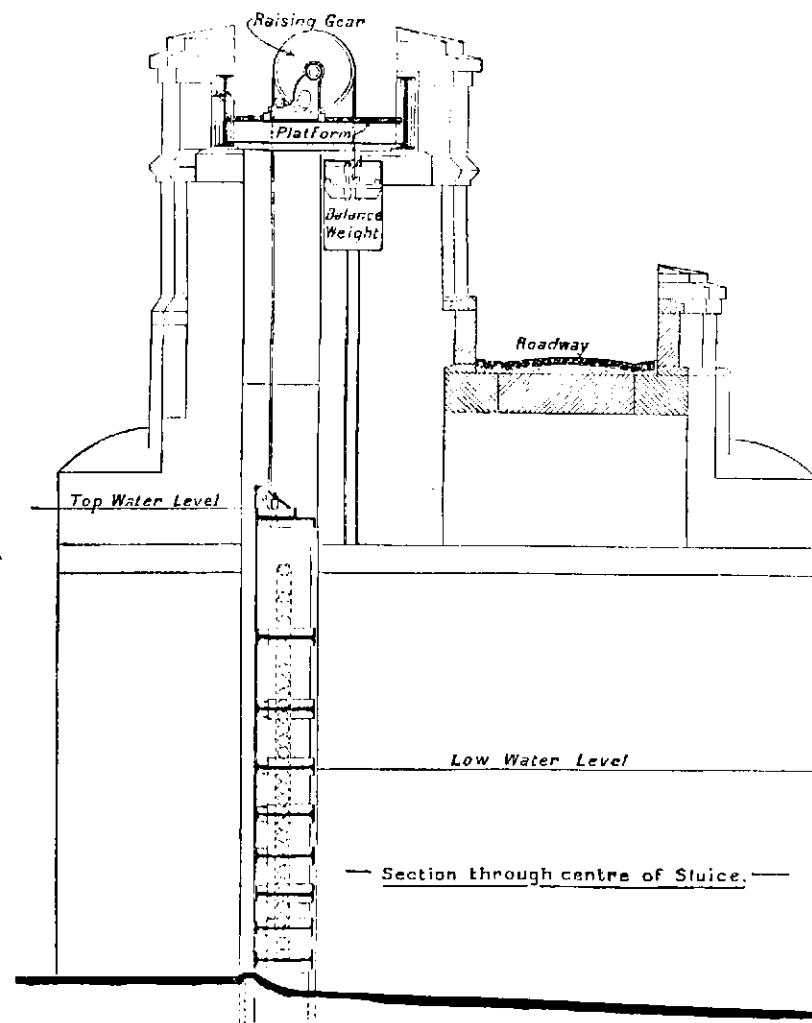


FIG. 497A.

of railway traffic. The science should demand more attention in the near future, due to the Royal Commission on Canals and Waterways, who have just issued their bluebook. The work, however, remains in the hands of the engineers holding appointments under river conservancy and navigation boards, more than municipal and county engineers. Reverting to

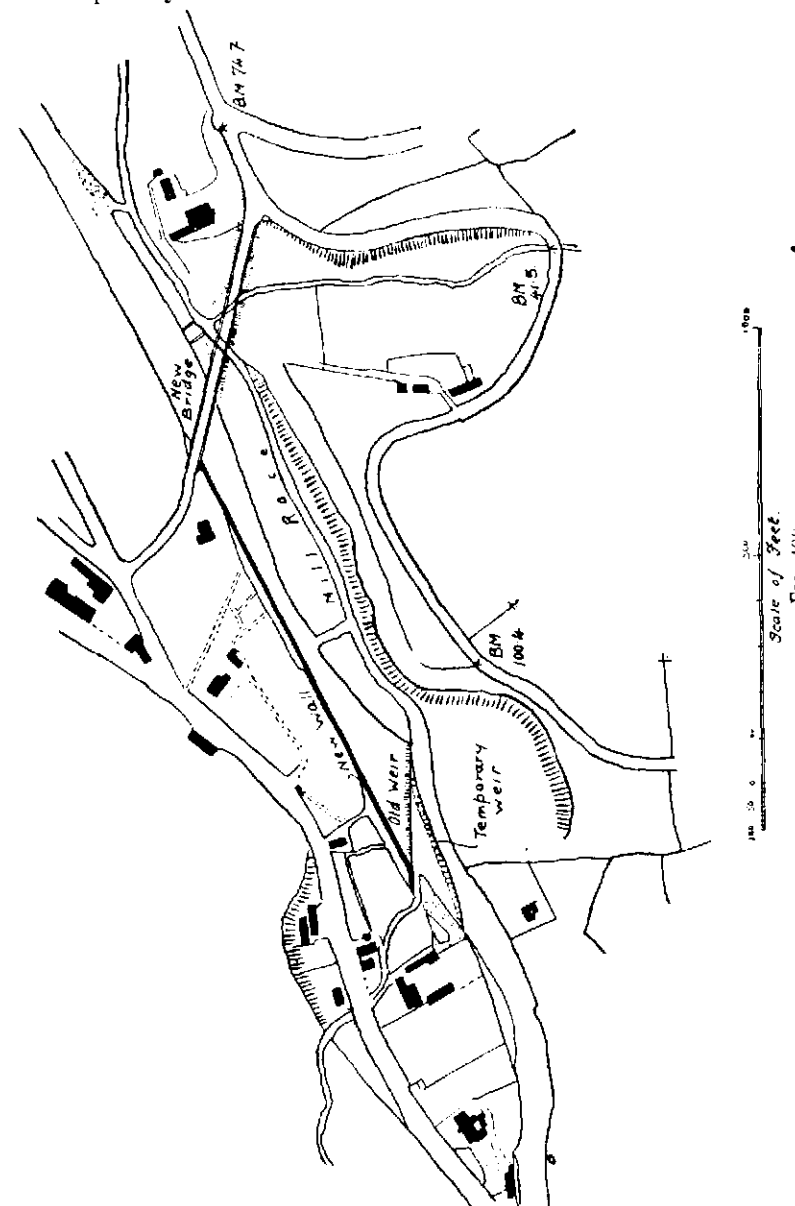
An ingenious form of self-acting weir, which was originally on the river Irwell at Thostlenest (now removed owing to the construction of the Manchester Ship Canal), is that shown in fig. 497*f*. The shutter is shown closed. The weir being self-acting, tilts over slightly on a light flood coming down the river.

It could, however, be specially worked by hand through gearing when a sudden flood rose. Sudden floods are very likely to occur on rivers which pass over impermeable strata, partly because of such impermeability, and partly on account of the high ratio of fall which such rivers usually have. That is to say, we shall have rapid rises and falls of big floods. On flat, sluggish rivers, however, the reverse will take place. The winter, too, is usually the time for high floods, but a very heavy rain may cause them to occur in summer, and if the ground has got very hard owing to a continued drought, the damage, especially in a hilly district, or in a river in which old cumbersome bridges and sluices exist, may be very considerable. Hence the necessity for providing means of letting off at very short notice large volumes of water. The effects of land drainage upon floods and river régime will be considered in Chapter XXIII, under Land Drainage. All rivers left to themselves will be more or less liable to floods. At the same time it has been proved that light floods have benefited a district by reason of the fertilising silt brought down, especially by winter floods. The practice is, of course, carried on artificially on a large scale in Egypt on the Nile.

As an example of river works, in accordance with our previous reasoning, we give the following:—

Fig. 498 shows a general plan of the works. Owing to a disastrous flood in 1905, which inundated the surrounding country, which is thickly populated with the dwellings of the poor, the old weir shown on the drawing was practically washed away. It was of rubble masonry, set in hydraulic mortar. It was afterward attempted to rebuild this in concrete, but through indifferent engineering this scheme failed before completion, letting the party who was responsible for the payment of the works into large sums of money. It was then temporarily repaired by means of cement bags placed across the river as shown. The primary object of the weir was to supply the town electric generating station by means of the mill race; but the other expedients having failed, the writer's opinion was consulted. Hitherto the supply at the generating station has only been of an intermittent nature, and quite inadequate for the Victor turbines there installed. The writer thereupon conceived the idea of storing up a large quantity of the day flow for use during the night, which was the time of heavy load. This, coupled with the growing necessity of a new bridge and connecting road where shown, led him to design the works as shown in figs. 499, 500, 501–502, and 503. We before gave the section of the ground on the proposed works in fig. 39 as an example of practical levelling, and the reader should refer to this in connection with the works we are describing. The bridge itself would be built of granite masonry, and the weir of concrete. In the weir were four culverts, each 8 ft. wide and 7 ft. high in the centre. The sectional area of each culvert is about 50 sq. ft., giving a total waterway of $4 \times 50 = 200$ sq. ft. The culverts are specified to be built of a special brand of hard Stafford bricks set in cement. At maximum flood-level they would each be able of discharging about 35,000 cub. ft. per minute, equal in all to a total discharge of 1,400,000 cub. ft. per minute when the sluices were full open, which would meet any demand likely to come upon them. When, however, this amount was exceeded, the

water could flow over the crest of the weir, which would discharge also a considerable quantity. The sluices at the power station would also constitute



another outfall. The culverts were designed to take Stoney's patent submerged sluices, which would be operated by crab gear and counterbalance on the bridge itself. The ordinary discharge would of course take place over the

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CIVIL ENGINEERING PRACTICE.

weir crest in the usual manner. The weir which would be built up with the bridge is of concrete, laid in layers on the heart, but the crest and front slope would be faced with coursed masonry in the manner which before was advised. Owing, however, to the extra amount of land submerged to make a storage reservoir a new cut had to be made into the mill race, as shown on the block plan, and a retaining-wall erected on the north bank of the river to prevent inundation of the surrounding lands. This was filled up at the back with the surplus excavation for the foundations, and the debris from the removal of the old weir and sand bags to clear out the river channel (see fig. 498). The roadway was formed together with the paths in the usual manner.

Of course, what we have said regarding the cross section and design of

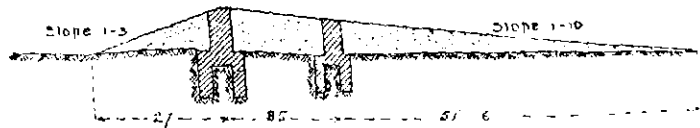


Fig. 505.

weirs in general must not in any way be taken as binding; like most engineering works, each case is considered on its own merits. Two forms of weir, having exceptionally flat slopes, and specially adapted to very large rivers, are shown by figs. 505 and 506. They would be formed of dry-stone, but the walls running through them, called nucleus walls, would be of solid masonry or concrete.

Owing to the courtesy of Messrs Ransomes & Rapier, I am able to produce to the reader fig. 506a.

This illustration shows a range of four sluices made for Siam, for the purpose of controlling the flow from a river, and also keeping out tidal water

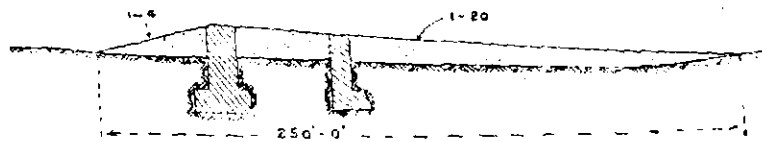


Fig. 506.

from the sea. The sluices are therefore constructed to withstand pressure from either side.

In order to reduce obstruction in the river and provide as much discharge area as possible, the piers are constructed of steel sections instead of masonry. The whole of the structure is designed for bolting down to a solid mass of concrete.

Each gate is 20 ft. wide $\times$ 18 ft. 6 in. high, with a lift of 18 ft. 6 in., and is counterbalanced. The operating gear is carried on overhead girders at the top of the piers.

At each end of the range of sluices steel bulkheads, with piers similar in construction to the sluice piers, are provided, so that an earthen bank may be brought up close to the openings, and so avoid the usual abutments.

We said a few words previously upon Groins, and gave examples of such

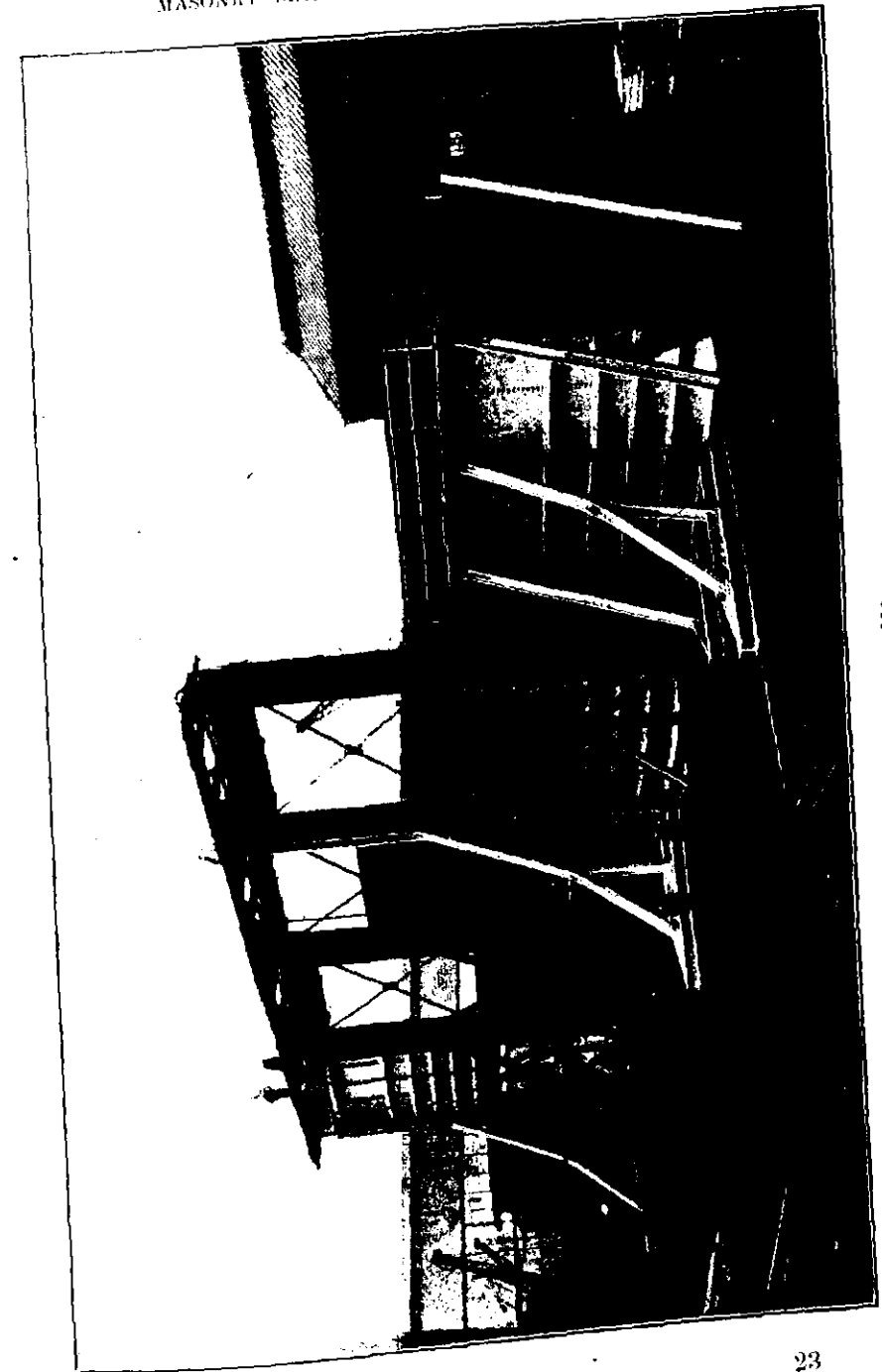


Fig. 506a.

as would be suitable for coast defence. We will now investigate the question one which concerns a municipal engineer a little further.

* "The great struggle for mastery is always going on between sea and land but only now and again, when a heavy gale occurs, is this warfare of the sea impressed on the public mind. The sea is making inroads on all our coasts, but more especially on the east coast. The coasts where high cliffs face the sea are more liable to erosion than low-lying land, and one may quote as an example some recent enormous falls of cliff at Dover. For this class of erosion

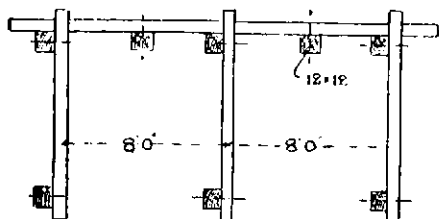


Fig. 506b.

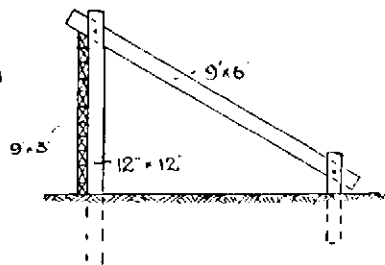


Fig. 506c.

very little can be done towards arresting the inroad of the sea. In low-lying shingly places, however, numerous efforts have been and are still being made to counteract erosion. The principal localities of active coast erosion are between Bridlington and Spurn Point, and along the East Anglian coast. It has been calculated that the average loss of land between Bridlington and the Humber for many years was at the rate of from $2\frac{1}{2}$ yds. to 3 yds. per year, and that a width of two miles has been washed into the sea since the Norman

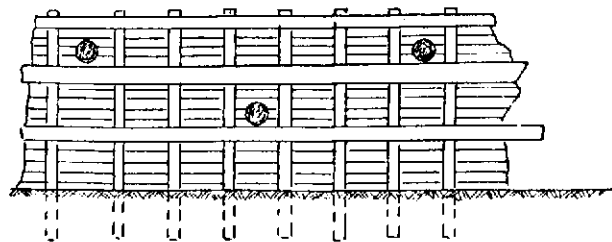


Fig. 506d.

invasion; but of late years strenuous efforts have been made to arrest this erosion. Bridlington itself during the past thirty-five years has spent more than £70,000 in protecting its foreshore. The method that was adopted was to construct timber walls backed with chalk, and concrete and masonry walls. There is a continuous movement of shingle up and down the east coast, and in order to arrest this the most effective method has been found to be timber groins.

"The Board of Trade system of groins that is adopted between Kilnsea and Spurn Point and at the Point has been very successful, and has resulted in the accumulation of an immense quantity of shingle and sand. This system is shown in figs. 506b and 506c. Piles 12 in. by 12 in. of Baltic fir are

* *Building World.*

driven 10 ft. into the clay, and sheeting of 3-in. planks is fixed to the piles by $\frac{3}{4}$ -in. bolts. On the lee side this sheeting is supported by 9-in. by 6-in. struts. The groins are generally 100 yds. in length and 200 yds. apart, and cost about £5 per yard-run. The bank of shingle that is formed by these groins makes a complete protection to the shore, which had become very dangerous, owing to the constant removal of shingle. Groins should not be

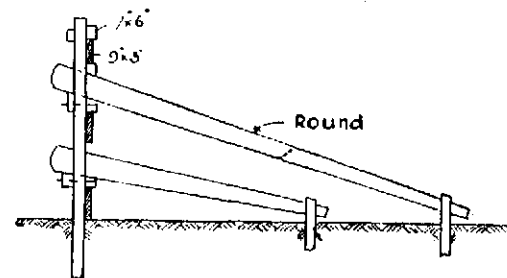


Fig. 506e.

built above high-water level of spring tides, or they will only cause excessive wave action and erosion.

"Various efforts have been made in East Anglia to deal with coast erosion in some definite united way, but up to the present in vain. Each locality acts on its own devices, but the erosion is yearly becoming so serious that it is hoped the Board of Trade may see its way to bring the matter before Parliament, and create some 'Coast Guarding' commission in a true sense of the word."

Figs. 506d and 506e give further details.

CHAPTER XXII.

HYDRAULICS.

THE term Hydraulics is the name given to that branch of study which affects the engineer's practice concerning the behaviour of liquids. It includes hydrostatics, the behaviour of water in a state of rest, and hydromechanics, water in motion. It is very important that every engineer should most thoroughly understand this branch of his studies; and with the excellent literature, both British and foreign, now published, he should find no difficulty in obtaining that knowledge. The only thing which may puzzle him is to decide which is the best of the many excellent treatises. Of course the present chapter does not intend to be an exhaustive treatment of the subject, but more a practical guide to the more everyday problems of the flow of water, and as an introduction to the remaining chapters of the book, which all more or less concern the flow of water.

A fluid is a body which offers no permanent resistance to change of shape. This is important.

The most important law in hydraulics is that of Pascal, which states that "fluids transmit pressure equally in all directions, and any increase of pressure at one point of a liquid is at once transmitted to every other point." Take fig. 507, a vessel having pistons held by light springs, and assumed to be quite devoid of friction. Say one of them is 1 sq. in. in sectional area, and it has a load of 10 lbs. on it. All the others will at once be subject to a pressure of 10 lbs. per sq. in. if they are on the same level. If not, of course the extra weight of the water will make a difference. Now everyone knows that water finds its own level. This is because pressures at two points in the same horizontal plane are equal. Upon this law depends the phenomena of suction, which is really balancing the air pressure. For instance, take the ordinary mercurial barometer in which the mercury will stand to a certain height, about 30 in., due to atmospheric pressure.

Again, a body when immersed in a liquid is buoyed up by a pressure equal to the weight of the liquid displaced by the body. This is the principle of Archimedes. It is for this reason that we can obtain the volume and density of a body by weighing in air and water (see Testing Road Stones).

A gas differs from a liquid in the lesser extension of its particles, and a gas is very compressible, and liquids practically incompressible. That is the difference between them. Gases also have very little weight, but the common idea that they have none is quite erroneous. For instance, the barometer will fall, according to its height above sea-level, approximately 1 in. for every 1000 ft. Now when water is at rest it comes to a level surface. This is called the free surface. As we go down below it, so the pressure exerted by

the potential energy of the liquid gets greater. It is called the *head*, and is the actual pressure in excess of that of the atmosphere. Now if water has a head of 1 foot, the corresponding pressure which it will exert is 434 lb. per sq. in. For comparison, 1 in. of mercury exerts a pressure of 491 lb. per sq. in.

Now when we measure down from the free surface, a distance = h and w = the weight of water per cubic foot = 62.5 lbs. Then p = pressure per square foot

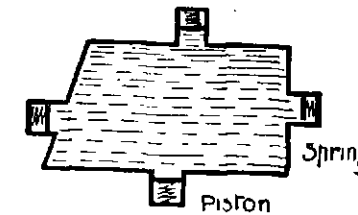
$$= wh \quad (275)$$

and obviously if a = a certain area at this point,

$$= wha \quad (276)$$

= total pressure on that surface.

This, of course, applies to level surfaces, and in horizontal surfaces the centre of pressure or point where the pressure is concentrated coincides with the centre of gravity. When the surface is vertical or sloping the conditions become changed. We consider the pressure as a uniformly varying stress,



Horizontal Section

FIG. 507.

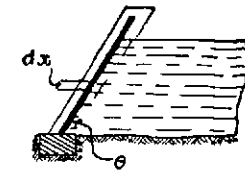


FIG. 508.

and its intensity at any given point varies as the distance of that point from the water surface, always measured in a line with the plane of the surface under consideration. Then consider fig. 508. A plane surface is immersed in water. It is inclined at an angle θ to the horizontal. Take any strip dx — x = distance from C of G of this strip to the surface.
 $w = 62\frac{1}{2}$ lbs. for water.

y = length of the particular band dx , of which we are supposing an indefinite number have been taken. Now the intensity of pressure on the strip

$$= \rho = wx \sin \theta \quad (277)$$

and the pressure on the whole surface

$$P = w \sin \theta \int xy dx \quad (278)$$

Then obviously the intensity of pressure (the mean over the whole surface)

$$= \frac{P}{S} \quad (279)$$

and S

$$\int y dx \quad (280)$$

and the distance of centre of pressure from this surface

$$= X_0 = \frac{\int x^2 y dx}{\int xy dx} \quad (281)$$

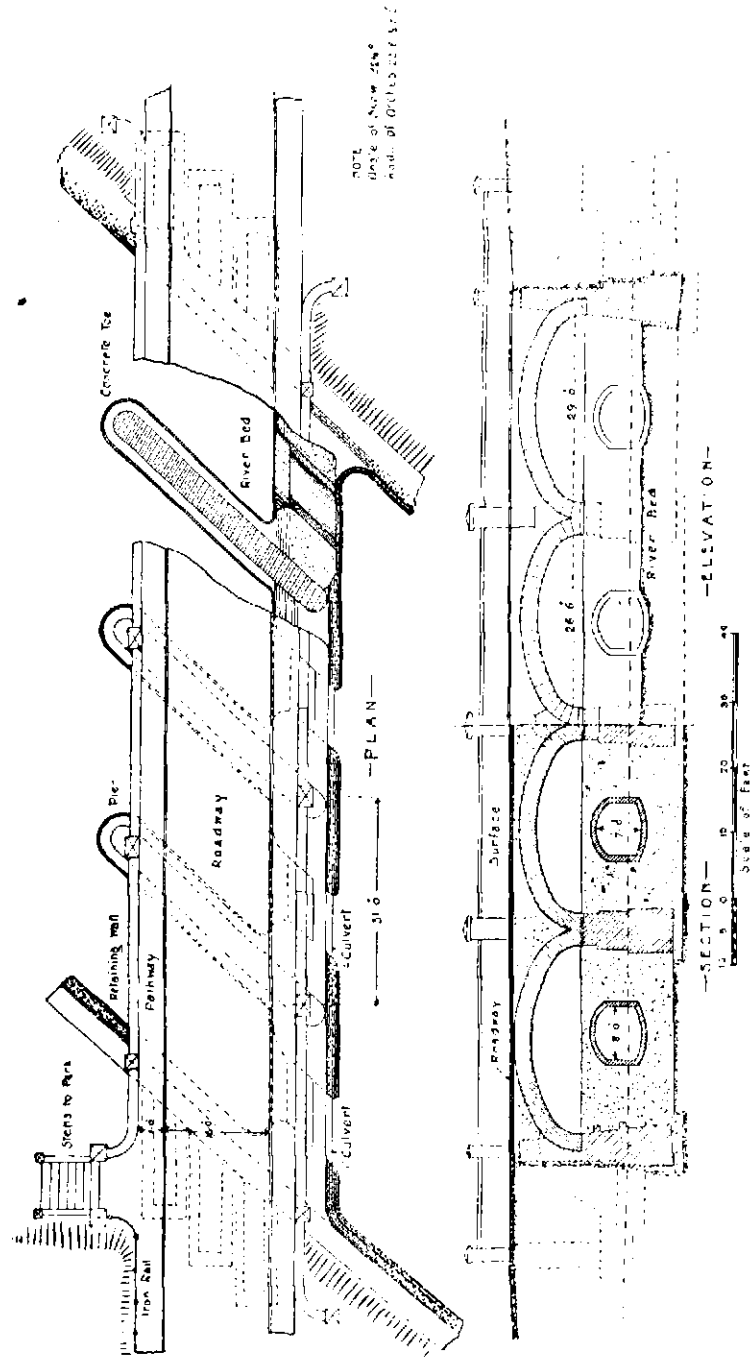


FIG. 492, 500, and 501.

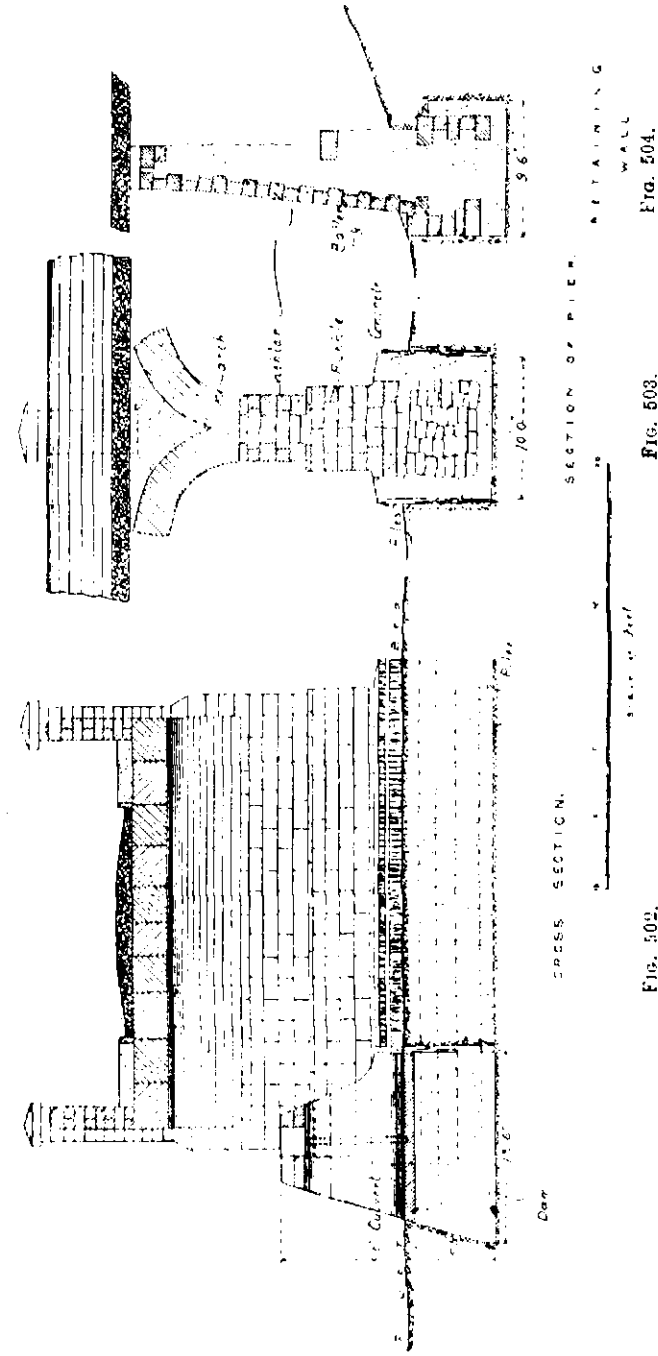


FIG. 502.

FIG. 503.

FIG. 504.

For the reasons stated respecting the resistance of a liquid to change of shape, we find that in any fluid the internal stress will be normal; on the other hand, in a solid body it may be normal, oblique, or shearing. Before going to the more practical and everyday side of the problem we must first investigate the theory of Hydraulics. The first point is the principle of continuity of flow. That is to say, if v = velocity in any tube, a = a sectional area, then Q the quantity of fluid passed

$$= Q = av \quad (282)$$

Now we have already spoken of head in reference to water. If we have water under a given pressure, and it be allowed to flow quite unrestrained, it will do so with considerable velocity according to the "head." If we know the head we can calculate this velocity. Let h = head and v = velocity in



Fig. 509.

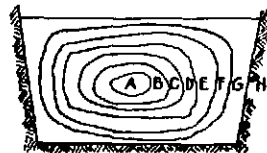


Fig. 510.

ft. per second, $g = 32.2$ and is a formula for the effect of gravity upon falling bodies (*vide* text-books on Mechanics). Then

$$v = \sqrt{2gh} \quad (283)$$

but usually written as

$$v = 8\sqrt{h} \quad (284)$$

H is of course the *mean* head. Conversely

$$h = \frac{v^2}{2g} \quad (285)$$

this quantity $\frac{v^2}{2g}$ being known as the velocity head. These equations give us the unrestrained flow from such orifices, which is purely theoretical. Various coefficients of contraction, discharge velocity, etc., have to be considered, and they are exactly what we are going to consider; but at the same time the reader should bear in mind that upon the relations between head and velocity all problems relating to the flow of water depend.

Now probably the most important practical hydraulic calculation an engineer will have to make is that of gauging the flow of small streams for water supply and power purposes. Take any rectangular channel, as in fig. 509; in the centre we shall have decidedly more velocity than at the sides. Say the flow at the centre was 5 ft. per second. Then at the sides we may reasonably expect a flow of 3 ft. per second, and at intermediate points 4 ft., etc., as shown on the diagram. Now take the mill race in fig. 510, in which, by means of a current meter, we have found the several velocities as tabulated below in feet per minute, the letters in the diagram referring to corresponding letters and figures in the table. The areas between the contours must then be obtained preferably by a planimeter.

Then we want to find the simplest way of calculating the mean velocities

between two contours. To do so add the velocities of two contours and divide by 2. For instance, say the velocity at A was 280 ft. per minute and at B 270. The mean velocity between the two contours will therefore be $70 + 280 \div 2 = 275$ ft., and the quantity flowing in this section will be $275 \times 12.41 = 3412.75$ cub. ft. per minute. The mean velocity between B and C is 265 ft., and the corresponding quantity flowing will be 9757.3 cub. ft. per minute. Therefore the total quantity flowing will be found by calculating the velocity and quantity between each two contours, and adding together the quantities thus found.

Velocity at	Feet per Min.	Areas between	Sq. Ft.
A	280	AB	12.4
B	270	BC	36.81
C	260	CD	69.25
D	250	DE	95.42
E	230	EF	116.10
F	210	FG	135.50
G	190	GH	144.94
H	0	...	...

This is the manner in which streams and rivers of considerable quantity are measured, and where a gauge would be inconvenient.

The current meters which are used, of course, vary in form. Some register on dials which are an integral part of the gauges, others have means of electrical contact with the surface. Of the former type, the Moore, Revy, and Woltman meters are examples, and of the latter, Harlacter's and Deacon's. It is most essential that they be very exactly calibrated by experiment, and equations and coefficients *not* relied upon.

Frequent observations are necessary at different points, both vertically and horizontally. At the same time the variations of water surface-level must be very carefully noted. We can then plot the discharge rates at various levels. These diagrams are useful to the engineer who has automatic float gauges. Of course accumulations of weeds will impede the use of current meters. Submerged floats have then to be resorted to. Quite the simplest and most effective form is the rod float, made out of a rod of wood. They have a cork at the top and a lead weight at the bottom; they should be in length equal to from .90-.94 of the average depth of the stream in question. In all cases, however, the cross section of the stream must be very accurately obtained by sounding and levelling. The sounding, of course, has to be done from a boat. This may be kept in position by a line from the shore where there is no traffic on the stream, or if this is impracticable, by taking observations to three points on the shore by means of a theodolite, and plotting the angles so found on paper, from which, if the points are also plotted in their exact positions on the paper, the boat's position at any observation can be found. The current meters would be, if possible, suspended from a bridge. If not, two poles with a strong steel wire joining them would be resorted to. Along this wire, by means of an accurately made winch, the meter could be made to travel to any position. A counter on the winch would give the distance out on the stream the gauge was. Regarding the velocity, it must not be supposed that it will be constant at all depths any more than at all points on a horizontal section.

* 12.4 = the sectional area in square feet of the stream at this section.

The maximum velocity will occur at about $\frac{1}{6}$ of the depth below the surface, while the mean will occur at about $\frac{2}{3}$ of the depth. The latter quantity will vary, but not very much. The former is practically constant when the velocities are plotted in a systematic curve; like so many other scientific curves it will approach the parabolic. If, however, we only have the surface velocity we can calculate the average velocity from a certain formula, say V_s = surface velocity and V = mean velocity

$$V = .7051V_s + .003V_s^2 \quad (286)$$

V and V_s being in feet per second.

We will now come to the all-important question of gauging streams. We do not propose to deal with rainfall here, but will do so in our chapter upon Waterworks, as being more suited to that chapter. It will be sufficient to say that when an engineer starts upon a waterworks scheme he very carefully consults rainfall records. Then, having the area of his proposed watershed, he will know the exact amount of water it *should* yield. Various losses, how-

ever, such as evaporation, cause the actual yield to be considerably below such expectations. It is for this reason that the supplying streams and rivulets have to be carefully gauged. They mostly take the form of temporary gauges made of planks. They must be read at least three times a day to give accuracy, and the readings to be of any practical value must extend over a

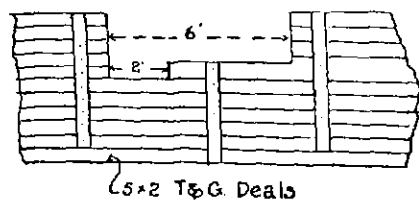


FIG. 511.

considerable period, and compared with the actual rainfall at the time. They must be of such a form and in such a position that the water is well ponded up behind them, so as to allow it to come to comparative rest. The early engineers, of course, had to calibrate such gauges by means of tanks of known capacities, and it is as the result of their persevering experiments that we have now very accurate formulae which do away with such troublesome procedures. The size of the gauge is determined by the drainage area. It must pass the greatest floods. That is the reason that most gauges have two or three notches cut in them, so as the small flows may be accurately registered. The building of such a gauge to give good results requires a little care. In the first place, the water has to be altogether diverted by a temporary channel. If this is impracticable, cofferdams must be formed taking half the stream at a time.

The conditions to be specially observed in construction are as follows:—

- (1) The ends must tail into the banks at least 3 ft., and be puddled with clay.
- (2) The gauge must be set truly level, and well puddled at the bottom.
- (3) On the downstream side of the gauge there must be a stone apron of 18 in. of stones, set at least 6 in. below the lowest notch.
- (4) End walls of masonry are preferable to mere puddle.
- (5) It must be very strongly strutted.
- (6) In loose ground it should be founded upon piles, and sheet piles driven at the upstream side to prevent the puddling blowing out under pressure. The bearing piles would be 4 in. to 6 in. square.

Now bearing in mind the conditions set forth, a very suitable form of

age is given in figs. 511, 512, and 513. The planking should be 2 in. or more, and all joints tongued and grooved. They may be square on the top, rounded, bevelled (on the outside), or the top may be a thin iron plate. The first thing to do when using such a gauge is to find the level of still water above it. A suitable apparatus is shown in fig. 514, and consists of a round brass rod AB, having a hook of brass wire fixed to it and brought to a sharp point at C. The rod AB slides vertically in the tube, which is securely fixed to the support. It is split along one side and has a vernier cut upon it;

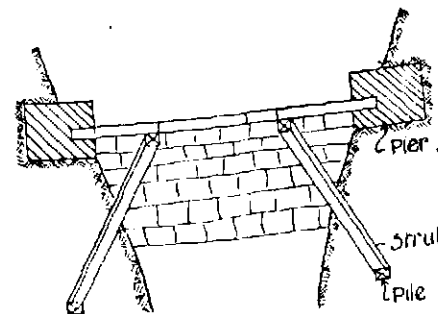


FIG. 512.

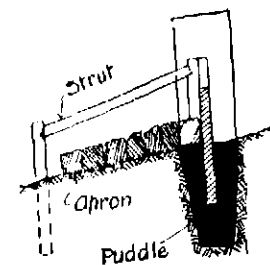


FIG. 513.

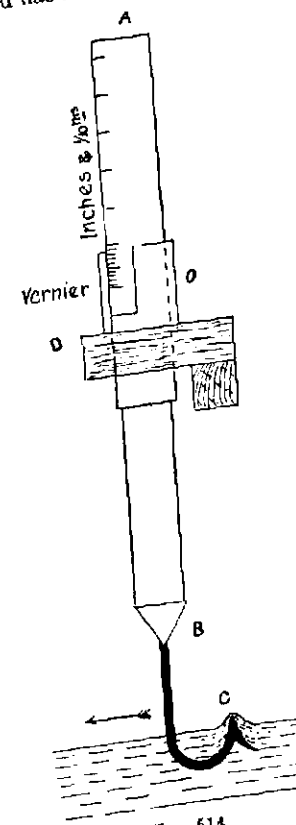


FIG. 514.

on the rod AB is a scale of inches and tenths. In use the rod AB is adjusted so as the hook just causes a small pimple to appear on the water surface. The instrument should be placed some distance back from the gauge, and must be very carefully set so that the top of the hook is exactly dead level with the sill of the gauge when reading zero. The depth of water flowing over the gauge which is ascertained by this apparatus affects the flow, inasmuch as the greater the depth the greater the quantity flowing, but also (as we before explained) the head due to the depth causes the velocity to be proportional to the square root of the head. Having set up the apparatus in a truly accurate and satisfactory manner, it remains to calculate the flow by means of a

satisfactory formula. We want to find the depth of water flowing over it = H and the breadth of the weir L .

There are various formulæ in general use. Those which are simplest, provided they give accurate results, are necessarily the most satisfactory.

Q = quantity in cub. ft. per second.

Then first we have Mr James B. Francis' equation,

$$Q = 3.33 \left(L - \frac{H}{5} \right) H \sqrt{H} \quad (287)$$

or if we can only measure the depth on the sill in inches conveniently = h ,

$$Q = .0801 \left(L - \frac{h}{60} \right) h \sqrt{h} \quad (287a)$$

These equations assume a double end contraction.

If there is none $Q = 3.33 LH \sqrt{H} \quad (288)$

$$Q = .0801 Lh \sqrt{h} \quad (289)$$

Again, there is Mr Thomas Hawkesly's formula,

$$Q_g = \frac{1}{2} L h^3 \quad (290)$$

In this case Q_g = gallons per second.

But we have only been dealing with rectangular notches, as shown by fig. 511. It is often convenient not to make them that shape, but to make them in the form of the Cippoletti weir, as in fig. 515. The formula to use is

$$Q = 3.367 LH \sqrt{H} \quad (291)$$

$$Q = .081 Lh \sqrt{h} \quad (292)$$

Where any considerable amount of gauging has to be done on the same weir, which is usually the case, the value $H \sqrt{H}$ or $h \sqrt{h}$ is usually determined for a series of values and kept for reference. Our next point to consider is the flow in open channels and conduits, and also culverts, which do not usually run much more than half full. When we treated rivers and streams we had special reference to rough channels, such as mill races having ordinary earthen banks. We now are going to consider the flow in masonry or concrete channels, of a more or less smooth nature, wherein the coefficients are considerably different, and where the cross section can be arrived at with a nicety, and also the velocity. The first thing to do is to find out what is termed the *hydraulic mean depth*. This is equal to the cross-sectional area of the stream $\div$ length of the wetted sides (or wetted perimeter). For instance, in any channel whose width is 3 ft. and depth of water 2 ft., the sectional area = $2 \times 3 = 6$ ft. sq. Again, the wetted perimeter = $2 + 3 + 2 = 7$ ft. $\therefore$ hydraulic mean depth, or as it is usually written HMD, = $\frac{6}{7} = .8571$. In a circular culvert running full or half full HMD = $\frac{1}{2}$ the radius. Because if we have any triangle whose base = the circumference of a circle, and whose height = the radius, the sectional area of the circle = that of the triangle, and the area of any triangle = base $\times$ perpendicular height. $\therefore$ HMD when pipe is full = area $\div$ circumference, and half the area $\div$ $\frac{1}{2}$ circumference when half full, both equal to $\frac{1}{2}$ radius.

Then if H_p = hydraulic mean depth, and f = twice the fall per mile $\div$ the conduit, V = velocity in lineal feet per second,

$$V = \frac{1}{11} \sqrt{H_p f} \quad (293)$$

This is Eytelwein's rule.

Now what we have said leads us to anticipate a form of cross section for an open channel which would give the best results in practice. That proposed by Mr Neville is the form which appears to satisfy these conditions, and is shown by fig. 516, bounded by three straight lines, all tangents to one semi-circle described upon a diameter which is the surface-level of the stream under ordinary flow.

Then we have certain coefficients.

One is $N = \frac{A}{H_p^2} \quad (294)$

A being the sectional area.

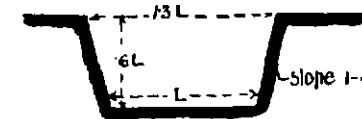


FIG. 515.

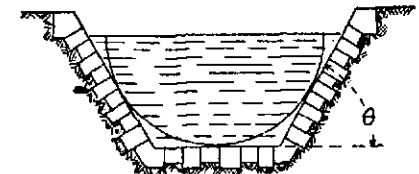


FIG. 516.

The following values of N are used in practice—

$$\left. \begin{array}{ll} N = 6.2832 & \text{for a semicircle} \\ N = 8 & \text{,, } \frac{1}{2} \text{ square} \\ N = 6.928 & \text{,, } \frac{1}{2} \text{ hexagon} = 4\sqrt{3} \end{array} \right\} \quad (295)$$

and for the proposed section in fig. 516—

$$N = 4 \left(\operatorname{cosec} \theta + \tan \frac{\theta}{2} \right) \quad (296)$$

In all the four figures $H_p = \frac{1}{2}$ the greatest depth.

Then there is another coefficient = f , and for open conduits—

$$f = .0074 + \frac{.00023}{V} \quad (297)$$

(Weisbach's equation)

and for cast-iron pipes $f = .005 \left(1 + \frac{1}{48 H_p} \right) \quad (298)$

V = velocity ft. per second.

H_p in Equation 294, always in ft.

Now in an open conduit the loss of head

$$= H_L = \frac{f L}{H_p} \cdot \frac{V^2}{2g} \quad (299)$$

where L = height of the channel between the two points over which the loss of head is calculated, and knowing this value of H_L the virtual inclination

$$= i = \frac{H_L}{L} \quad (300)$$

d = diam. of pipes in inches, and S = the fall, which, say the fall was 1 in 240, would stand in the equation as $\frac{1}{240}$. For instance, in the 3-ft. sewer,

$$V = 13.725 \sqrt{36 \times \frac{1}{240}} = 5.40 \text{ ft. per second.}$$

Sewers are often made egg-shape, however (*vide* Main Drainage). The top water-level should not rise above the centre of the top circle (*vide* fig. 519). To calculate the flow in such a sewer, the first thing to do is to calculate its sectional area. The part above the largest diameter is obviously equal to half the circle described on that diameter, but the part below it is found by equation

$$= D^2 \times .755813 \quad (311)$$

and if d and S are the same values as given for the circular sewer,

$$V = 15.3 \sqrt{dS} \quad (312)$$

$$Q = .08 \sqrt{d^5 S} \quad (313)$$

Thus in a 2 ft. 6 in. egg-shape sewer the working sectional area would be $2.5 \times 2.5 \times .755813 = 4.6872$ sq. ft. It is interesting to know that when any

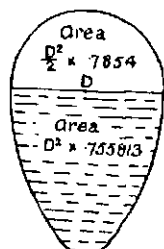


FIG. 519.

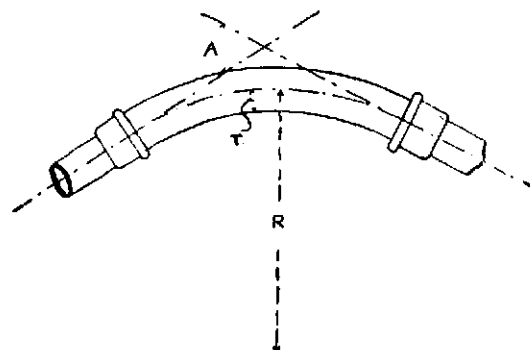


FIG. 520.

liquid such as sewage enters a main sewer or culvert from a branch, it will have to travel along the main for a certain distance before it acquires the velocity of the main. From any of the equations given we can find the velocity in the main. Now the branch stream will suffer a loss of head

$$= h = \frac{V^2}{2g} \quad (314)$$

Then if we place the value of V , referring to the main culvert, on top of Equation 314, our result will be a certain number of feet head. This is the loss of head which the stream suffers. Say the fall of the sewer was 1 in 240, we can reason that, because there is a gain of 1 ft. head in every 240 ft., there will be a certain distance where the head so lost will be regained. In the same way, if we have a sudden rush of water at V ft. per second, the same reasoning will show a certain distance over which the velocity will keep up (all the while gradually diminishing till it falls into synchronism with the

We now come to deal with the flow of water in pipes. These pipes which we are going to consider are pipes for supply of water for domestic or power purposes; they will usually be made of cast iron or sometimes of welded or riveted steel, and are assumed to be running quite full under pressure. Now if we have a pipe of diameter D ft., under a pressure or head of H ft., its total length being L , also in feet, the velocity in that pipe in feet per second is

$$V = 50 \frac{\sqrt{DH}}{L} \quad (315)$$

Now the flow of water in a pipe is of course dependent upon the friction in that pipe, and the friction is proportional to—

1. Length of pipe.
2. Inversely as diameter.
3. As velocity squared.
4. As roughness.
5. Independent of pressure.

Now when the velocity V is found, and we know the sectional area, the discharge in cub. ft. is easily found. Multiply this by 6.25 and we have gallons.

Then if we have

G = gallons discharge per minute,
 L = length of pipe line in yards,
 d = diameter of pipe in inches,

we can find the loss of head due to friction by the equation

$$H = \text{loss of head in ft.} = \frac{G^2 L}{(3d)^5} \quad (316)$$

Or if it is a pipe line for a power scheme, the loss of power would be a more important quantity.

Then if H_1 = the loss of energy in foot-lbs. of every pound of water passing through the pipe, L = length, and D = diameter of pipe in feet, and V as already calculated,

$$H_1 = \frac{.0007 LV^2}{D} \quad (317)$$

and we know that there are 33,000 ft.-lbs. of energy in a horse-power. Now the resistance to flow in any pipe is, as we have pointed out, as the square of the velocity. So, say we have a pipe line carrying a certain quantity of water, and we are going to lay a branch to take, say, $\frac{1}{4}$ of the quantity. If D = diameter of the main, what will be the diameter of the branch? Its value—

$$= d = \sqrt{\frac{D^2}{6}} \quad (318)$$

D and d being in inches, but the main would continue, or reduced in size only $\frac{1}{4}$, that is a 12-in. main would become a 10-in. one. A loss in pipe lines which would especially affect power questions is that due to bends, for which we have the following equation, giving the loss of head in ft. :

$$= H = .0135 V^2 \left(\frac{A}{180 L} \right) \quad (319)$$

The equations of course depend upon the frictional resistance of the sides of the channel. Of course there is no special form of conduit to be adhered to; it is mainly a matter of local consideration for the engineers. In rectangular channels the best proportion is perhaps a width of 2 and a depth of 1 for the stream. That will give the greatest HMD for a given quantity of material used in construction. The following considerations will deserve attention.

1. Small conduits are liable to freeze more than large ones. This entails their being covered.
2. Snow is a disadvantage.
3. Open conduits for domestic supply are only permissible in open country where free from contamination.
4. Conduits having a velocity of more than 3 ft. per second must have an artificial facing.
5. The best side slopes are $1\frac{1}{2}$ -1 and 2-1.

The form which aqueducts take are defined as—

- (1) tunnels, (2) open channels, (3) cut and cover, (4) raised aqueducts.

Other equations which may be used in comparison with those given in reference to open channels are when S = the length in which the surface falls one unit, that is to say, if the fall is known to be 2 ft. per mile, $S = 5280 \div 2 = 2640$.

Then V = ft. per second.

$$V = \frac{88.51(\sqrt{H_p} - .03)}{\sqrt{S - \text{hyp. log } \sqrt{S} + 1.6}} \quad (301)$$

The equation would be confined to moderate velocities of 2-3 ft. per second. When still more accuracy is required, we want to take into account the coefficients of roughness of the material of which the channel is built. It appears as N in equations, and its value is—

- (a) Earth channels (good order) .020.
- (b) Brickwork and ashlar masonry, .013.
- (c) Ordinary rivers and rills, .028 about

Then a suitable formula is that of Kutter—

i = the hydraulic slope of the water surface.

H_p , N , and S as before, and V = ft. per second.

$$V = \left\{ \frac{\frac{1.811}{N} + 41.6 + \frac{.00281}{\sin i}}{1 + \left(41.6 + \frac{.00281}{\sin i} \right) \frac{N}{\sqrt{H_p}}} \right\} \times \sqrt{H_p S} \quad (302)$$

It may here be stated that all the formulæ given will give accurate results in practice, and they err slightly on the right side, that is, they under-estimate the flow. Again, in addition to formula 302, we have an equation to find the mean velocity when the surface velocity is found, because finding the surface velocity is a fairly simple matter, because such bodies which have practically the same specific gravity as water itself will float very nearly immersed. The writer finds such simple expedients as an orange or gutta-percha ball to be very handy for the purpose. Now we stated before that the theoretical velocity due to head = $V = 8\sqrt{h}$ was subject in practice to certain modifications. Say we have a tank with a pipe at the base a certain mean height below surface-level of 9 ft. The theoretical velocity would be

$V = 8\sqrt{9} = 8 \times 3 = 24$ ft. per second. But the coefficient of discharge for such a pipe is .505. $\therefore$ the actual discharge would take place at $24 \times .505 = 12.12$ ft. per second.

Now in such cases it is usually the practice to make the connection between such a pipe and the tank by means of a bell-mouth; and if we construct such a bell-mouth as shown by the fig. 517, we shall reduce this coefficient to unity, and get practically the theoretical velocity at the commencement of the pipe line. Say now we have a pipe with a contraction as in fig. 518, similar to the venturi tube we are going to describe in a later chapter. If Q = discharge in cub. ft. per second,

$$V = \frac{Q}{A_2} \quad (303)$$

and the effective area at $a - ca$ (304)

while the value of c = coefficient of contraction,

$$c = \frac{.618}{\sqrt{1 - .618\left(\frac{a^2}{A_1^2}\right)}} \quad (305)$$

Hence the velocity in the part $a = rV$ (306)

and $r = \frac{A_2}{ca}$ (307)

Now on reservoirs the most essential piece of the work is perhaps the waste weir. Its construction we shall describe under Waterworks.

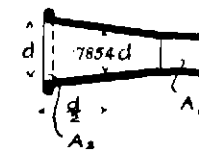


FIG. 517.

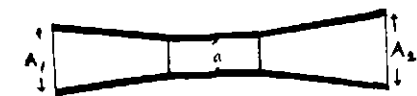


FIG. 518.

The length of such a weir in feet is usually made equal to 38 ft. for every 1000 acres of drainage area, but a better calculation is

$$L = \frac{Q}{5\sqrt{d^3}} \quad (308)$$

L being in feet and d being the maximum allowable depth in ins. to which the water may be safely allowed to rise above the crest. Q cubic feet per minute, and

$$Q = 4.81 \sqrt{ld \times ld} \quad (309)$$

l and d being length and depth of water flowing over weir in feet.

Take now, for example, a large circular sewer, bearing in mind that sewage will not flow so easily as water; at the same time the sides of the sewer would doubtless be of a dirty, rough nature.

Taking this into consideration, we use the formula

$$V = 13.725 \sqrt{dS} \quad (310)$$

V being velocity in feet per second.

A = angle shown in fig. 520, while L appears in the following table, when certain values of $\frac{r}{R}$ are considered, viz. the ratio of semidiameter of the actual pipe to the radius of curvature.

$\frac{r}{R}$	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
L	131	138	158	206	294	44	66	98	14	20

Now all pipes must be laid with at least a certain gradient to provide power to overcome friction. Such a gradient is known as the hydraulic gradient, and the formula to find it is

$$\frac{2.3V^2}{d} \quad (320)$$

This will give us the required fall in feet per mile, which roughly pans out at 10 ft. per mile in ordinary cases, viz. a gradient of 1 in 528. Perhaps the hydraulic gradient would warrant a little further explanation. Say we have a reservoir and a pipe line from it. Say at any point on the pipe line the pressure recorded was 50 ft.

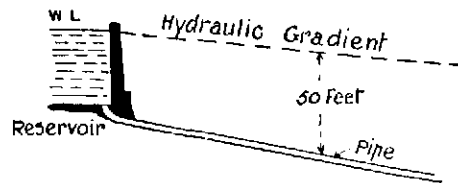


FIG. 521.

Then if this line and the reservoir were plotted correctly on a section, vide fig. 521, the line joining the top water-level of the reservoir and a point vertically 50 ft. above the pipe line at the point in question would be the hydraulic gradient. This, of course, assumes the pipe to be

running full, but full only; no pressure at the top of its circumference. So if a pipe has not a gradient equal to the hydraulic gradient, there would be a point on the pipe where we should have no pressure, and its distance in feet from the reservoir for any pipe would be

$$L = \frac{25001DH}{V^2} \quad (321)$$

D = diam. in ft.

But provide the pipe with the proper gradient and it will run on for ever!

Having regard to the weight of cast-iron pipes, tables for use in estimating are published, but if these were not handy we can find out the factor by means of a simple equation also. Given D = the outside and d = inside diameter in inches. W the weight in lbs. per yd. of the pipe is equal to

$$W = 7.85(1^3 - d^3) \quad (322)$$

while if we have flanges, 2 flanges = 1 ft. of pipe, or if the joints are spigot and faucet, 1 spigot and 1 faucet = 1 ft. of pipe also.

In water schemes one of the most essential calculations for mains of any importance is their size. Too small a pipe causes shortness of supply, while pipes too large, besides being useless expense, tend to contaminate the water by long standing. A little discretion is necessary.

Say we have to supply W cub. ft. of water per minute, the total length of pipe being L ft., its diameter = d ins., and the pressure = H ft.

$$\text{Then} \quad d = .531 \sqrt[5]{\frac{LW^2}{H}} \quad (323)$$

The fifth root of the equation will of course be solved by logarithms and by dividing by 5.

If, however, we have to distribute the quantity by means of a number of small pipes in preference to one large one, and their proposed diameter is D and d ins., the number to be provided will be equal to

$$N = \frac{\sqrt[5]{D^5}}{\sqrt[5]{d^5}} \quad (324)$$

We shall now deal with what is termed the contracted vein, or the *vena contracta*. It is the principal retarding force to the flow of water through orifices of thin plate, weirs, and other orifices, but its effect in pipes is nil, friction taking its place. For thin plates its value = $c = .618$. When, however, a thin plate notch is set up in a weir whose total breadth is B ft., the width of the notch being b ft., its value is

$$.57 + \frac{b}{10B} \quad (325)$$

Again, for a vertical sluice in a rectangular channel $c = .70$; but if this sluice is inclined away from the flow from 35° – 60° , its value becomes $c = .75$. Now in the sequel we shall confront certain quantities such as h^3 and h^4 . To solve the latter, logs must be used, multiplying the log of h by $\frac{4}{5}$, but for the former we can solve it easier by a table of squares and cubes. For instance, find a square whose value is nearest to h , and opposite this we find a value in the cubes column. This, then, is the approximate value of h^3 , which is really the cube of the square root of h .

Now we mentioned before that in gauging streams a pond of still water was absolutely essential. Sometimes, however, it is a very difficult matter to obtain this, in which case we have to measure very carefully the velocity of approach, and then calculate it. For instance, if V_a is the velocity of approach in feet per second, the height of water on the sill will be approximately equal to

$$H_a = \frac{V_a^2}{2g} \quad (326)$$

This is not the actual height, but the height for our calculations. A useful equation for such weirs is

$$Q = 5.35cbh^3 \quad (327)$$

b being breadth of weir in feet, h = head of water in inches, c being the coefficient of contraction, either .618 or the value found by Equation 325. But, as before stated, we may have to find a value for H_a . Find out by means of careful levelling what H would ordinarily be, and then calculate H_a , and instead of Equation 327 use the following—

$$Q = 5.35cb(h + H_a)^3 - H_a^3 \quad (327a)$$

In both cases we shall have $Q = \text{cub. ft. per minute}$. But if by any means we cannot find V_a , we only have to compute it by finding Q from Equation 327, and dividing by the sectional area of the stream at which h is measured. Instead of rectangular notches, triangular ones have been used as in fig. 522. The formula to use, which will give a value of $Q = \text{cub. ft. per second}$,

$$Q = \frac{1}{15} B \sqrt{2g} H^3 \quad (328)$$

the values of B and H being shown on the diagram. Due to end contraction, however, we have to modify the equation, its new value being

$$Q = c \frac{1}{15} B \sqrt{2g} H^3 \quad (328a)$$

c being a coefficient = .59 when the notch is a 90° one. On the other hand, when $B = 4\frac{1}{2}$ (very flat), $c = .62$.

Again, when the notch is 90° we can have another modification, that is,

$$Q = c \frac{1}{15} \sqrt{2g} H^3 \quad (329)$$

The edges are assumed to be levelled off sharp.

Take now a circular hole in a tank discharging freely into the atmosphere.

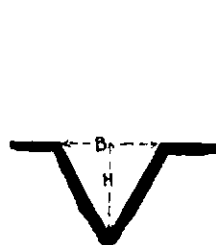


FIG. 522.

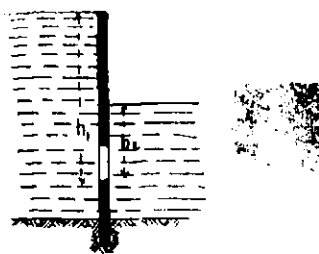


FIG. 523.

If its diameter = d in ft. and h = head also in ft. to surface of still water the discharge

$$Q = 3.9 d^2 \sqrt{h} \quad (330)$$

We now have to treat a submerged rectangular opening such as the sluice illustrated by figs. 492 and 493. Referring to fig. 523, we see two values h_1 and h_2 , and the discharge

$$Q = cb \left(h_1 + \frac{h_2}{2} \right) \sqrt{h_1 - h_2} \quad (331)$$

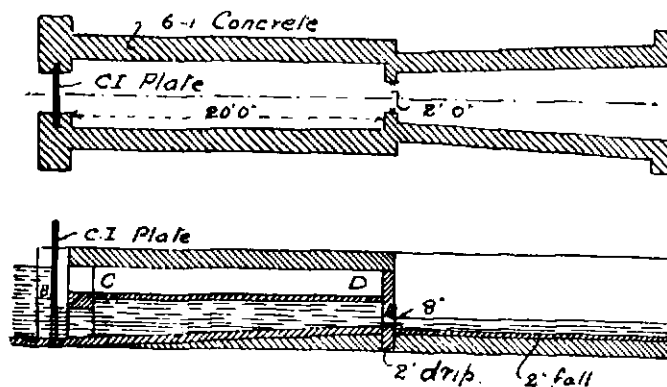
c being a coefficient equal to .70 if the sluice stands vertically, or .75 if inclined to the horizontal (see previous reasoning). Now it will be seen that it is a comparatively easy matter to transpose any of the foregoing equations to give other required values. For instance, such an equation as $Q = 8.025 c A \sqrt{h}$ would be transposed so as to give A so—

$$A = \frac{Q}{8.025 c \sqrt{h}} \quad (332)$$

and so on.

When treating the subject of water pipes we assumed these to be new pipes, fresh from the works, the bore being quite smooth. This is usually the

case when we are considering calculations. In use, however, they are liable to corrode. Corrosion is practically the same in all pipes irrespective of diameter, and is usually considered as making the diameter 1 in. less, that is, we assume $\frac{1}{2}$ in. of corrosion all round. To prevent the rapid corrosion of pipes (to altogether avoid it is as yet an impossibility), they should always be coated with Dr Angus Smith's patent solution, or they may undergo a treatment called the Barff process, which consists of heating the pipes to a cherry-red and exposing them to the effects of highly superheated steam. The superheat must, however, not be too great, because it has been found in steam superheaters for power purposes that at very high temperatures disassociation of the steam will take place, and the iron will rapidly corrode to red oxide by the action of the free oxygen. In the actual process the oxygen of the steam will combine with the iron and form a protective coating of magnetic oxide of iron. In our chapter on Weirs we mentioned certain formulae which would receive consideration in reference to



FIGS. 524 and 525.

the same. When a weir is placed across a river to raise the water-level it will effect that level some distance back by raising it above the actual weir crest. Now if we know the discharge over the weir = Q , and the breadth of the weir = b , then the height on the sill to which the water will rise will be

$$h_1 = \left(\frac{Q^2}{7b^2} \right)^{\frac{1}{3}} \quad (333)$$

This assumes the weir not to be drowned. The calculations referring to the amount of backwater so produced are rather too complex for a work of this class. For further information see Rankine's *Civil Engineering*, p. 690.

A peculiar form of measuring weir used extensively on irrigation works is illustrated in figs. 524 and 525. The opening B = the orifice of discharge constant in width, but its height may vary. The total rise of the floor space is 16 in. There is a stone lintel at A and flooring CD . The bottom of A is set level with the underside of B and 4 in. below the surface-level and 2 in. above the sill of the sluice. The tail chamber starts by being 4 in. wider than the orifice and tapers outwards. It has a 2-in. drip and a 2-in. fall to its ends. To work the gauge we must have 8 in. difference of level on the two sides of the

sluice. They are of Italian origin, and not more than 6 ounces of water are allowed to pass through any one opening. The formulae for discharge are

$$V = c\sqrt{H} \quad (32)$$

$$Q = AV \quad (33)$$

$c = 4.96$, H = head to centre of orifice from top level. Of course the primary object of these gauges is to do away with measurements, their construction being such that the head will not vary, and consequently the flow remains constant once for all.

We shall now explain what is commonly known as the venturi law, applied to a very efficient form of water meter which is described in Waterworks Distribution. Referring

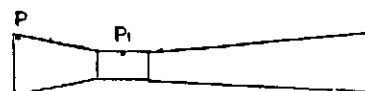


Fig. 526.

fig. 526, which is a line diagram of a venturi meter, we see a narrow throat and 2 expanded cones, the downstream one being considerably longer than the upstream

one. Now when water flows along this pipe we should find that the pressure due to its head at P was not the same as that at P_1 , in fact it will be somewhat greater. Call the pressure at $P = P_1$ and at $P_1 = P_2$.

Then $P - P_1$ gives us what we call the venturi head.

The principle of the meter in its practical form is that the diminution

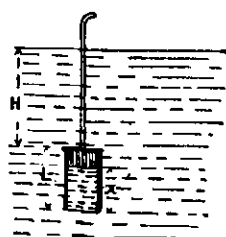


Fig. 527.

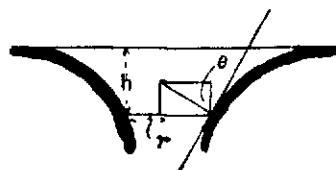


Fig. 528.

of area of the pipe causes a corresponding increase of velocity and at the same time a loss of head. Then if H_v = the venturi head, the theoretical velocity in the throat

$$V = 8\sqrt{H_v} \quad (34)$$

for which a coefficient of say .77 would be used, but it is usual to determine a coefficient for each meter by experiment. Then the discharge in gallons per minute

$$= .0141672 Vc \quad (35)$$

d being throat diameter in inches and c the constant.

Now take the following hydraulic problem as illustrating some of the previous reasoning. Say we have a diving-bell in fig. 527. The bell assumed to be 10 ft. long and the top water-level in the bell is known to be 50 ft. below the surface. We want to know, when the air is entrapped in the water, how high the water will rise in the bell. Now if the depth 50

= H , and the length of the bell = L , and h = the height of the water barometer = 34 ft., the distance

$$x = \frac{HL}{H+h} \quad (338)$$

$$= \frac{50 \times 10}{50 + 34} = \frac{500}{84} = 6 \text{ ft.}$$

In some hydraulic problems, especially those of centrifugal pumps and turbines, we have to deal with what is called the vortex. A vortex may be either free or forced. In the free vortex the energy of the fluid is the same at all points, but in the latter it is different at different places. Fig. 528 shows a diagram of the free vortex. At any point which is a distance h below the surface, the velocity due to the head is obviously

$$V = 8\sqrt{h} \quad (339)$$

but in addition to this we have a horizontal force equal to

$$\frac{mV^2}{r} \quad (340)$$

being the radius at the point in question, m being the mass of a particle of

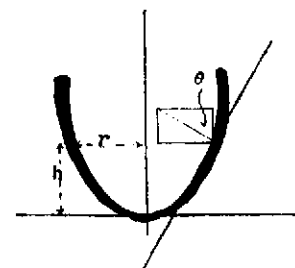


Fig. 529.

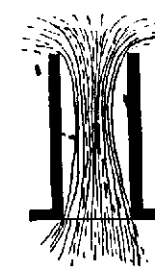


Fig. 530.

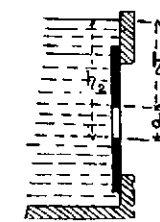


Fig. 531.

water flowing at that radius. When these are plotted in their proper directions, the resultant of the two makes an angle θ with the vertical, and

$$\tan \theta = \frac{2h}{r} \quad (341)$$

Hence, for this reason, we have the vortex chamber or volute in a centrifugal pump, to convert the kinetic energy into pressure energy. A simple example of a free vortex is the whirl which water usually takes when flowing down the waste pipe of a lavatory basin, noticeable by everyone. Now the forced vortex is really the free vortex turned upside down, and is parabolic in form. It is shown in fig. 529, and the same reasoning applies to it.

It was before stated that the pressure due to any head of water was wah . This is called the statical pressure. The reaction pressure

$$= 2wah \quad (342)$$

and is the force which acts upon a turbine vane or similar appliance. But say, by means of a suitable guide, we have a double reaction so $\left(\frac{P}{P_{12}} \right)$, we have a

pressure at $P_1 = 4 \text{ atm}$. In a moving vane, however, such as a Pelton wheel, we shall have certain modifications (*vide* Use of Water Power). Another thing to notice also is that when water flows down a pipe as in fig. 530, which would take place in the discharge pipe of a large flushing tank, the actual area of the stream will only be about half that of the actual pipe. The manner in which all water flows in fact depends above a certain limit upon the velocity, but below that limit upon its *viscosity*. The point of change is called the critical point. The former is called *eddy* flow, the latter *steady* flow. The critical velocity will vary as—

1. The ratio of viscosity to density.
2. Inversely as the diameter of the pipe.

Take now the case of water flowing through an orifice shown by fig. 531, when l = its length in feet, h_1 and h_2 measured in feet as shown,

$$Q = c d l \sqrt{2gh} \quad (343)$$

c has the following values:—

$l = 2 \text{ feet}$ $d = 1 \text{ foot}$ $c = .60$	$l = 2 \text{ feet}$ $d = 6 \text{ inches}$ $c = .615$	$l = 1 \text{ foot}$ $d = 1 \text{ foot}$ $c = .60$	$l = 1 \text{ foot}$ $d = 3 \text{ inches}$ $c = .63$	$l = 1 \text{ foot}$ $d = 6 \text{ inches}$ $c = .615$
---	--	---	---	--

and for holes 6", 9", and 3" square, $c = .60$, h is the mean of h_1 and h_2 . When both sides of the orifice are flooded, and the orifice is submerged, the above Equation 343 still holds good if $c = .67$ and h = difference of surface levels. It is, of course, quite necessary to have records of the heads on each side of the orifice in question. We will now conclude the chapter with two problems which will no doubt affect the engineer sometime or other during his career. For instance—

Problem.—The discharge from an 8-in. pipe whose length is $6\frac{1}{2}$ miles (=11,440 yds.), heads 190 ft. and 180 ft. respectively. Before one can ascertain the exact quantity of discharge it is necessary to make deductions from the head—i.e. (1) for velocity at entry; (2) for friction against the sides, or perimeter of the pipe. The formula undermentioned is the one in general use by engineers, and is known as "Box's formula," which makes allowance for friction, and its results are quite accurate enough for practical purposes. The loss of head due to velocity at entry may be neglected, as on a long length of pipe it would not make a great deal of difference.

$$\text{Let } G = \left(\frac{(3d^5) \times H}{L} \right)^{\frac{1}{4}} \quad (344)$$

where G = gallons per minute.

„ d = diameter of pipe in inches.

$$\therefore G = \left(\frac{(3 \times 8^5) \times 190}{11440} \right)^{\frac{1}{4}}$$

where H = head in feet.

„ L = length of pipe in yards.

Compute by logs.

$$\therefore \frac{\log \text{ of } 3 \times 8 = 24 \times 5 + \log \text{ of } 190 - \log \text{ of } 11440}{2}$$

$$(1) \log \text{ of } 24 = 1.3802$$

5

$$6.9010$$

$$+ \log \text{ of } 190 = 2.2788$$

$$9.1798$$

$$- \log \text{ of } 11440 = 4.0584$$

$$2) 5.1214$$

$$2.5607$$

$$\text{Anti log of } .5607 = 3637.$$

So the answer is 363.7
gallons per minute.

(2) By the same formula with
the head 180 ft.

$$\log \text{ of } 24 = 1.3802$$

5

$$6.9010$$

$$+ \log \text{ of } 180 = 2.2553$$

$$9.1563$$

$$- \log \text{ of } 11440 = 4.0584$$

$$2) 5.0979$$

$$2.5489$$

$$\text{Anti log of } .5489 = 3539.$$

Answer — 353.9 gallons per
minute; about $1/36$ less than
(1).

Calculating Diameter of Mains for Water Scheme.—The diameters for discharging a given quantity of water at the end of each branch with the

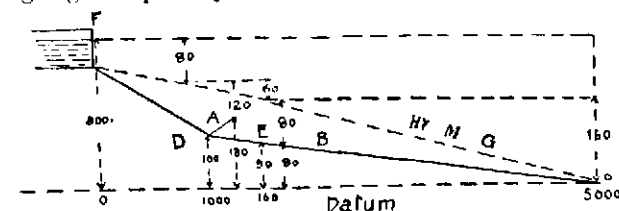


FIG. 532.

ends open cannot be determined by a direct method. The simplest method is to draw a section (see fig. 532) to a convenient scale; for instance, 100 ft. horizontal and 20 ft. vertical to 1 in. On this section set out an assumed hydraulic gradient. Assume the length of the branches to be from D to A 600 ft., and from E to B 300 ft., and the discharge 40 gal. per minute simultaneously at the end of each branch with their ends full open. The dotted lines in the section are the assumed mean hydraulic gradient of the mains and branches, which must be set out so that the pipes at no point are above the gradient. From the hydraulic gradients the frictional loss and the pressure can be scaled off direct without any calculation. The loss of head due to frictional loss at any point on the mains or branches is found by scaling off the distance that the mean hydraulic gradient is below the level line of the water surface in the reservoir. The pressure at any point in feet-head is found by scaling the distance that the pipe is below the hydraulic mean gradient, or in pounds per square inch by multiplying the height that the hydraulic mean gradient is above the pipe by .434.

The hydraulic mean gradient in the section shows a loss of head due to friction from the reservoir to point D of 80 ft., on the branch D to A a loss of 120 ft., from D to E a loss of 60 ft., branch E to B a loss of 80 ft., and from E to C a loss of 160 ft.

The quantity in gallons per minute that is to be delivered at D =

40 + 40 + 40 = 120 gal., and at E 40 + 40 = 80 gal. The quantity that has to be discharged and the head being known, the diameter of pipes that will be required in order to deliver this quantity at the different points of the system can be determined by Box's rule. By this rule the diameter of pipe required for delivering 120 gallons per minute at D will be $\sqrt[5]{\frac{120^2 \times 333 \cdot 3}{80}} \div 3 = 3 \cdot 01$ in., say 3 in. diameter, which is the nearest stock

diameter; from D to E $\sqrt[5]{\frac{80^2 \times 166 \cdot 6}{60}} \div 3 = 2 \cdot 36$ in., say 2½ in.;* from E to C

$\sqrt[5]{\frac{40^2 \times 1166 \cdot 6}{160}} \div 3 = 2 \cdot 171$ in., say 2¼ in.; diameter of branch D to C

$\sqrt[5]{\frac{40^2 \times 200}{120}} \div 3 = 1 \cdot 614$ in., say 1½ in.; and for branch EB $\sqrt[5]{\frac{40^2 \times 100}{80}} \div 3 = 1 \cdot 524$ in., say 1½ in.

The approximate velocities are calculated by the following rule $V = \frac{G}{2 \times d^2}$, where V is the velocity in feet per second, G discharge in gallon per minute, and d the diameter of pipe in inches. By this rule the velocity from the reservoir to point D will be $\frac{120}{2 \times 3^2} = 6 \cdot 66$ ft. per second, D to E 6 ft., E to C 3·948 ft., D to A 6·53 ft., and EB 8·889 ft. per second. With the ends of the pipes full open discharging the above quantities there will be no head or pressure at the discharge, the whole of the head being absorbed in friction.

* This size main and the two other sizes given, viz. 2½-in. and 1½-in. mains, are only given for sake of example, and are not usually stocked by makers; 2-in. and 3-in. are the usual sizes; some engineers never use less than 3-in.

CHAPTER XXIII.

LAND DRAINAGE.

THE drainage of land is to ensure that the land to be drained will be in a moderately dry state, not too dry in the wet seasons, but free from floods and soddenness or boggiess. It is an essential to agriculture.

The two great authorities upon the subject were Mr Smith of Deanston, whose system still retains the name of *Deanstonising*, and Mr Parks.

Both these gentlemen aimed at the same object, and with more or less success, but, strange to say, they differed materially in detail.

First of all we shall consider the system of Mr Smith. He advised that in all land drainage there should be frequent drains 10 ft. to 24 ft. apart. That these drains should be shallow, not deeper than 2 ft. 6 in. That parallel drains should be placed at intervals either in wet or dry ground.

That the minor drain should be laid on the slopes of, and the main in the hollows of, depression; and finally, that stone drains were preferable to tile drains or pipes.

It will be seen now how widely Mr Parks differed from these principles. He advised that the drains should be less frequent, say 20–50 ft. apart. They should be deeper also; say about 4 ft. He had no doubt a special object in view when recommending deep draining, which was a twofold one,—that is, to get rid of all surface water, yet keep the subsoil water below the limits of capillary attraction. He advocated the use of pipes for drains.

It is since the principles of land drainage were enunciated by these pioneers that the science has been fully developed.

In fact, companies have been formed for the purpose, with parliamentary assistance; for example, the Yorkshire Land Drainage Company, 1843. Subsequently legislation stepped in, such as the Public Moneys Drainage Acts and the Private Moneys Drainage Acts, which were passed during the last century, and naturally the science obtained great influence. Now the main object of land drainage is, of course, essentially to draw off surface water, and to prevent it lying stagnant. But it by no means ends here. That is only quite a primary object. All soils are more or less retentive. This retentiveness depends principally upon the closeness of the molecules which compose the soil. Obviously dense clay will absorb and hold more water than loam, and, in turn, loam will hold more than a sandy soil. Now when the soil is fully charged with moisture it naturally cannot hold any more. The result is stagnant water at the surface; and if this is not removed by a suitable system of drainage, it will naturally sooner or later evaporate. This makes the earth cold, because evaporation abstracts the heat from the earth. Another bad effect of stagnant water is that it shuts out the

warming effect of the sun's rays. Again, when the sun goes down and evaporation ceases, the surface water cools below the temperature of the subsoil; gravity then causes it to sink, and it will, in turn, cool the subsoil. Drainage will alter all this. The warm spring rains which give so much benefit to agriculture are not immediately evaporated as before, but will give up their heat to the soil; the result will be a harvest of quite 14 days earlier; in fact it has been proved so. Drained soil in midsummer would be about 2° Fabr. warmer than an undrained one. Now anyone who has studied rainfall statistics will see that not far off 77 per cent. of this when left to itself will be evaporated.* It is from the undrained clay soils that the greatest evaporation takes place. Loose soils do not lose very much as a matter of fact; $\frac{2}{3}$ of the rainfall is the average loss due to evaporation (= 66 per cent.). We can

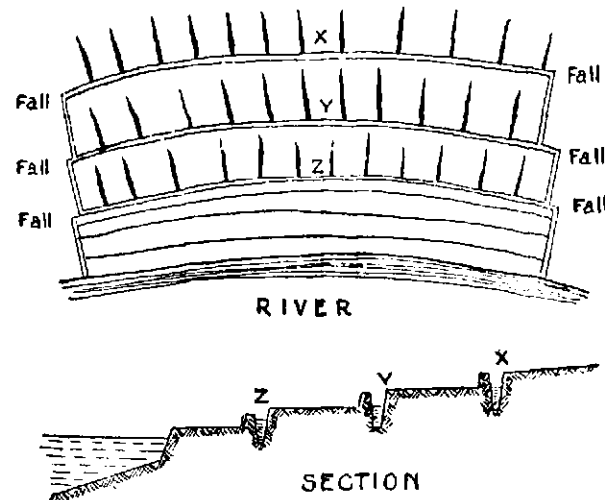


FIG. 533.

now safely conclude from what we have said that land draining is altogether an advantageous procedure when carried out in accordance with correct principles. The principal ones are as follows:—

1. Less evaporation.
2. Aeration of the soil.
3. Increased depth of nitrification.
4. Earlier harvest.
5. Facilitated tillage.
6. Improvement of pasture.
7. General healthiness of the surrounding lands. Of course in some places natural drainage will occur. It is called arterial drainage, and may be assisted, however, by artificial drains.

It will occur especially in oolite and chalk, mountain limestone and trap rock where near the surface. We, however, are dealing with lands which have no natural drainage and are excessively wet. The first thing the engineer has to do is to find out *why* they are wet. Having found out, then find out how much of the land is wet and how much dry. It will not *all* be bad probably.

* Or otherwise lost, as pointed out in a later chapter.

a frequent source of wetness is when porous soil obtains water from a distance. Such soil, of course, wants careful draining. Of course the only real way of finding out if a land does want draining is by experience and the natural unhealthy look of the land. The most common cause is an impervious subsoil. Where this is the cause, patches of the land will show a dark look in the spring during wet weather, which will disappear in dry. These patches occur more often than not on slopes. They will, of course, show the wet ground.

Regarding the time when such work could be most successfully carried on, the autumn and winter are the best times. May, however, may have to wait till opened by continued rains. It was a mistake to do the work by unskilled and female labour. This, in the author's opinion, is false economy, and that skilled labour should in all cases be provided for.

Low lands are perhaps the first kind which should gain our attention. Lands near river estuaries are the principal kind.

Provided they are above high-water level of ordinary spring tides, surface drainage is a simple matter, and consists of cutting ordinary minor and main drains in the natural hollows, providing small sluices for regulating purposes, the drains in plan being very similar to the branches of a tree.

The junctions only want careful attention; pitching with dry rubble may

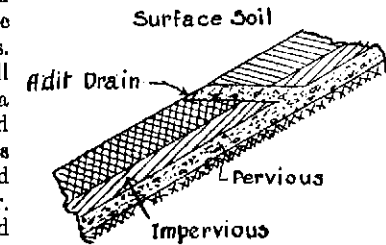


FIG. 534.

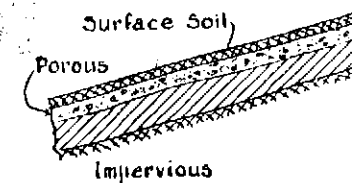


FIG. 535.

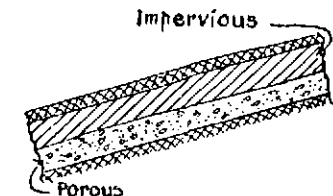


FIG. 536.

be necessary. Now, where the surface has a fair slope, the system of mains and branches no longer holds good. Catchwater drains, figs. 533, must be formed. By properly placed sluices, irrigation is made easy and efficient in dry seasons. Reservoirs may also be formed. Sometimes, however, an upland district is not so easily dealt with as may be supposed. Say we have a case as fig. 534. From the saturated substrata we have to bring the water to the surface by means of a special drain called an *adit* drain. Internal springs would receive similar treatment. Again, in all drainage works the capacity of the drains has to be considered when deciding upon their size. Drains too large add to expense; those too small add still more expense in rectifying the evil. Hydraulic laws must be taken into consideration, because drains having a good fall do not need such a large cross section as their flatter neighbours. Parallel drains should be aimed at, and right-angled junctions. These, however, must in no case be *opposite* each other. In addition to these considerations, the nature of the subsoil largely controls the position of the drains. For instance, take fig. 535. This will demand frequent drains, which would be well if carried *below* the clay. But when the clay is deep

it must not be interfered with. Any springs, however, found below the clay by boring must be tapped. But in fig. 536 the cases are reversed. The drains must be cut through the clay, and if not too deep, say 5-6 ft., right through both strata. The drains, which in this case must be of ample capacity, are very frequent on plan. When the case in question occurs over a natural depression, rapid floods are liable in the drain lying along the bottom

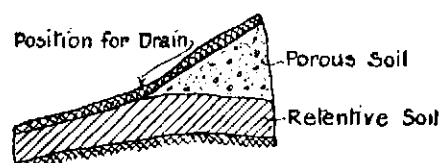


FIG. 537.

of such a depression. This wants due consideration. There, parallel main drains would be an advantage. Take the case in fig. 537. The main drain will be right along the base of the porous soil, and cut through the clay if not too deep. In fig. 538 three drains will be required as shown. Again, where we have on a hillside layers of clay and gravel alternately rising to the

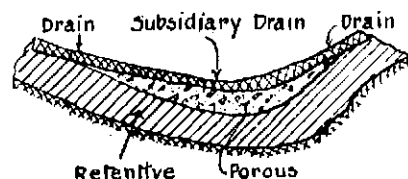


FIG. 538.



FIG. 539.



FIG. 540.

surface, at the foot of each gravelly layer is the correct place for the drain. Examples of surface drains are given by figs. 539-540, and of a large drain, fig. 541. This drain, with a fall of 18 in. per mile, would discharge 4641.4 cub. ft. per minute by formula $V = \frac{1}{11} \sqrt{H_p F}$, where H_p = hydraulic mean depth, and F twice the fall per mile. Drains running next to river beds must have

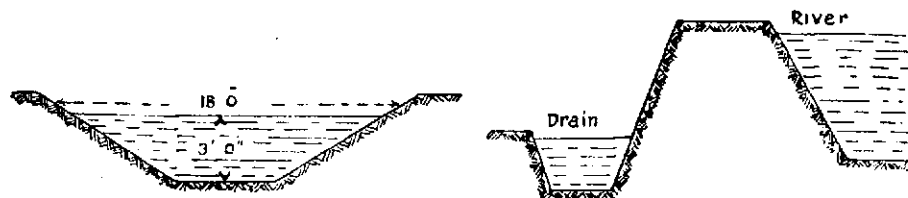


FIG. 541.

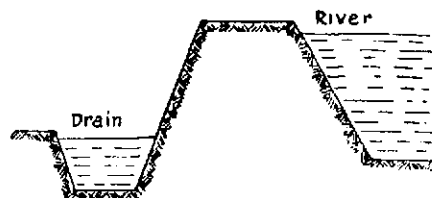


FIG. 542.

their beds below the waterline, shown by fig. 542. Pumping would be necessary.

We have previously been discussing open surface drains only. The rudest form is that of a simple furrow filled with brushwood or straw. They are only, of course, applicable to strong soils. Moreover, they do not, owing to their necessary shallowness, drain the body of the soil, and of course in a loose soil would become quite useless; and in most cases, both of arable and pasture lands, covered drains are preferable. The simplest form of covered drain is, of course,

the stone drain, fig. 543. Fig. 544 shows one built of cut or stratified stone, economical where stone is cheap. Fig. 545 shows a sole and tile drain, having loose stones as well. These drains are very satisfactory and economical. Perhaps the simplest of all, and at the same time the most permanent, is the drain made with circular fireclay field drains, usually 2 in.-3 in. in diameter and 12 in.-14 in. long. They are laid carefully on a firm bed with ordinary butt joints, and have a filling of stones around them. When speaking of culverts in reference to roads, we stated the damage likely to be incurred by tree roots. This holds good in regard to all land drains, which should be carefully laid away from all such roots. Vermin also have caused damage. Deep draining is now considered by most engineers to be an advantage, but the general levels limit this to a great extent. In any case they must be below likely damage by ploughing. On the other hand, there is a limit. For instance, in flat fenny districts, when the water-level of the ditches gets below



FIG. 543.

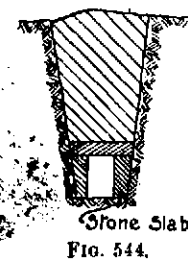


FIG. 544.

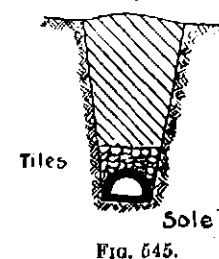


FIG. 545.

2 ft. from the surface, damage to crops is likely to result. The depth usually varies with the nature and condition of the soil.

Drains 2-3 ft. deep at 18 to 24 ft. apart will satisfy perhaps more requirements than any. But, as before suggested, the root of the evil may be quite below the level in the case of springs, and the only way to rectify the evil is to tap the springs, which will only suffice when the spring is fed from a

The usual method of tapping is by wells filled with stones. Now in forest lands, for previous reasons, underdraining is not resorted to. Main drains 3 ft. deep, 1 ft. wide at the bottom, and slopes 1 1/2-1 are the most advantageous. The minor drains are, of course, of smaller dimensions. The extreme distances between the minor drains would be 5-40 yds. Of course it is not only the requisite discharging capacity which will determine the size of a particular drain. The surface presented to the soil, by which it "sweats out" its moisture, so to speak, is an important factor. And it is for this reason that frequent drains in tenacious soils are of advantage. In clay soils, 50 in. is the minimum depth for covered drains. On the other hand, we must be careful and not carry drainage too far, and actually dry up the land, which would be caused by excessive deep draining in clay.

Now the draining of bogs puts another aspect on the question. Fig. 546 gives an example of a drain suited to the purpose. The drain is made of layers of dug peat. They have, however, the disadvantage of being liable to fall in, owing to the peat drying out. On the whole, however, it is well suited to its purpose when made during the summer. Tile drains are useless in bogs, owing to natural subsidence as the result of draining.

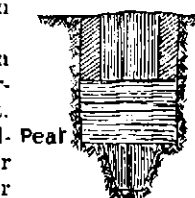


FIG. 546.

Now for all land drainage works undertaken by the engineer, like all other works, good maps or a survey are indispensable; careful levelling and accurate contour lines are most essential. The boring tools are then used upon the ground. The auger and punch are those mostly used. The lines of drains are then pegged out along the centre line, the width marked carefully off on each side, and the edges *lock spitted*. The use of boning rods is especially useful in this class of work, in which great accuracy is not aimed at. The actual cutting, however, demands reasonable care, the bottom being trowelled. Now on free soils it will be economical to ascertain by means of test holes the correct position for drains. Clays, however, must be uniformly drained. In free soils it is lowering of the actual water-level which is principally aimed at, and to let off the water without risk of stagnation, at the same time in a gentle and natural manner. In clays, however, the quicker the water is let off the better, because clays have such great absorptive powers. It may be here necessary to say exactly what we mean by clays and free soils. A free soil is considered to include sands, peats, gravels, and loams, and mixed earths in general.

Clay may be bare loam or gault. Any soils consisting of more than 10 per cent. of argillaceous matter are called clays when considering drainage questions. But although we have pointed out all the advantages of draining, yet there are certain disadvantages and necessary precautions to be considered.



Fig. 547.

For instance, they are very liable to cause floods on the lower lands. Suitable outfalls demand due consideration.

Now, as a direct result of under-draining, the water will run off the land in a shorter period. The result is the possibility of floods in the rivers and streams. The fact must attract the attention of the engineer for his consideration as to how far these effects will reach. Provision must then be made by deepening or straightening these streams to carry off all such excess water. Hydraulic laws must be again studied; in fact, reservoirs might be necessary to provide water to the land in dry seasons, and to regulate such floods. Shallow surface drains are to be advised, and those in lands liable to sudden floods are best made as in fig. 547 so as to confine the waters. The effect of drainage upon rivers is as follows:—

Case I.—A river with a low fall will remain longer in flood than when there was little or no drainage.

Case II.—A river with a high fall will have floods of less magnitude than previously, but they will rise and fall considerably quicker. But the river will not bring down so much silt, to the general advantage of any shipping at its mouth. In general, however, it may be said that drainage will cause increased floods and droughts on rivers. When ordinary conditions prevail, we may expect under-drains to start discharging about October in free soils, and in clay late in November, due to the absorptive power of clay. On the other hand, after continued winter and summer rains, they may run the whole year round. An example of a long surface drain is given in section in fig. 548. It was designed to take off quickly large quantities of water. So it was made 6 feet wide at the outlet at the lower end, and 4-5 ft. deep. The bottom width, however, diminished to 2 ft. at the upper end. It will be seen

that the gradients are fairly stiff, which would have caused (as will occur in a good many cases) too rapid a discharge altogether, with consequent damage to the drain. To avoid this, the gradients were carefully reduced and overfalls provided as shown. They were constructed of brickwork in cement, as shown by fig. 549. There were grooves for boards, so as to allow of the water being dammed back if so desired. Where a system of under and surface drains is in vogue, and the under ones discharge into the surface drains, these outfalls should be as few as consistent with prudence and economy, for the reason that these outlets when dry are very liable to chokeage.

The drainage of peat bogs is likely to be of interest to Irish engineers and county surveyors. Peat, which is so abundant in the bogs, originally being in the form of bog moss (*Sphagnum palustre*), owes its presence to successive growth and decay of the moss, hastened by the presence of water. The cross section of a bog is usually convex, the highest part being, strange to say, the wettest, and it is here the bog is in a semi-fluid state. Bogs become difficult to drain, owing to their usually receiving water from the surface and from springs. The first thing to do is to take levels, and note also the depth of the bog at each point. We will then be able to ascertain the *natural outlets*. The next thing to do is to make large surface drains, 5 ft. wide at the top and 3 ft. deep; the sides are very flat usually. Subsidence will then start when the drains are made in sufficient number at the same time. The drains are continually kept up to the same sizes. Finally the drains get so deep as to allow of their being sunk to discharge into the natural outlets, feeders being formed at right angles to them. This process carried on properly over a period from 2-5 years will cause the bog to become solid enough for carts. The minor drains would then be filled up, and the peat will gradually decompose on the surface into black mould. In red bogs, however, the whole mass of peat must be drained to the substratum, usually by a deep cut through the centre. Now the subject of peat digging has lately occupied the minds of engineers in Ireland for power purposes, but the only suitable state in which peat would be of any material advantage in practice for the generation of power by gas engines and producers is the form known as *anhydrous* peat; that is, peat in which all the water, or at least a very large percentage, has been got rid of. We cannot hope to go very far with the subject except simply intro-

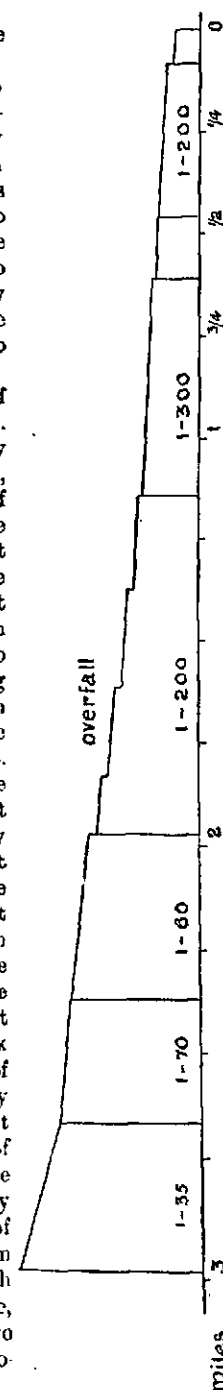


Fig. 548.

ducing it,* because it is entirely in its infancy as a source of practical power, but, from the able researches of Mr Tomlinson, an Irish engineer, much has been brought to light on the subject. The curve in fig. 550 and Table I. may be compared conjointly, which will afford much information in themselves.

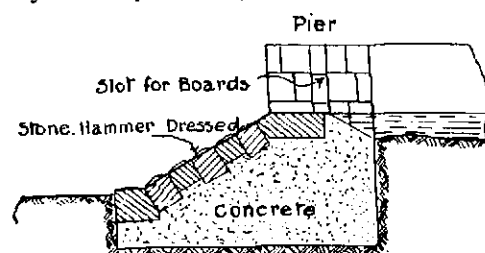


FIG. 549.

Now one ton of anhydrous peat, when suitably converted into gas, would give about 14,500,000 British thermal units of useful heat. But, in order that the power so obtained would be at a price able to compete with water-power, the peat would have to be obtained at 4s. per ton in the anhydrous state, which up to the present has not been made possible, and it is doubtful if it could be, as may be seen by the tabulated results in Table II. Machine working would slightly lower these figures. The very best by hand labour would be 6s. per ton.

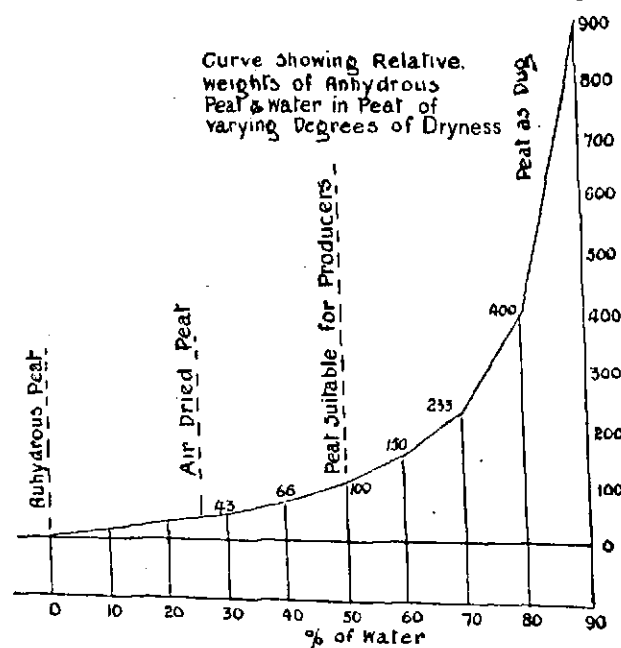


FIG. 550.

Fig. 551 gives a plan and fig. 552 a section of a typical Irish bog, and the method of successfully draining the same. Such a procedure gradually shrinks the bog and lowers its surface. Now one ton of peat in a virgin bog would

* See *Peat: its Use and Manufacture*, by Björling and Gissing; London, 1907. Also *Commercial Peat*, by F. T. Gissing; London, 1909.

TABLE I.

Anhydrous Peat in Lbs.	Water in Lbs.	Percentage of Water in Mixture.	Notes.
100	11	10	Air-dried peat.
"	17	15	
"	25	20	
"	43	30	
"	66	40	Suitable for use in producers.
"	100	60	
"	150	70	Peat as dug.
"	233	80	
"	400	90	

TABLE II.—COST OF PEAT GETTING AND DRYING—HANDWORKING.

Place.	Labour—Digging and Transporting to Drying Field.		Labour—Turning and Piling.		Total Cost in Pence.		Price realised per Ton Anhydrous.
	Pence per Cubic Yard.	Pence per Ton Anhydrous (Approx.).	Pence per Cubic Yard.	Pence per Ton Anhydrous (Approx.).	Per Cubic Yard.	Per Ton Anhydrous.	
Sparkær, Denmark.	3.5	46.5	0.65	8.6	4.15	60a	...
Moselund, Denmark.	2.5	33.4	0.80	10.7	3.3	40a	...
Triangel, Germany.	1.85	24.6	0.90	12.0	2.75	38a	...
Haspelmoor, Germany.	2.63	35.0	...	...	6.7	90	14s.
Raubling, Germany.	4.25b	56.5	2.45c	32.5	...	88	15s.
Bernau, Germany.	...	...	...	...	3.7d	65	...
Feilenbach, Germany.	...	...	...	...	4.12	73	14s. 6d.
Oldenburg.	...	...	...	...	...	...	...
Average.	2.94	39.2	1.2	15.9	...	...	...

TABLE III.—COST OF PEAT GETTING AND DRYING WITH DOLBERG'S PEAT-CUTTING MACHINE, HAND-OPERATED.

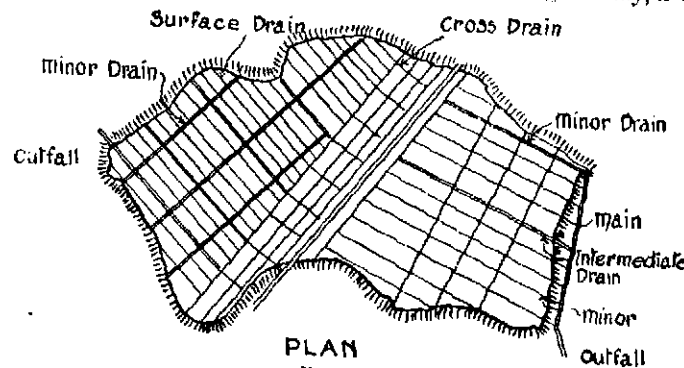
Ostrach.	3.6	48.0	1.9	25.4	5.6	73a	...

a These costs are for labour only. b Includes piling wet soils. c Includes stacking.
 d Add general expenses, 1s. 2d. per cub. yd. e This figure is probably excessive.

These figures are due to Mr Tomlinson, C.E. (Trans. Inst. C.E.I., vol. xxxv.).

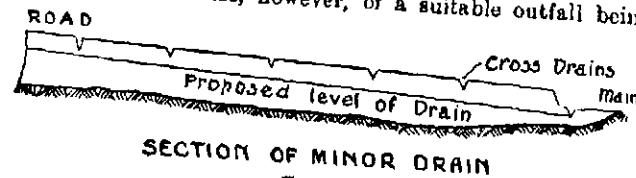
occupy about 15 cubic yards. After two years' drainage this would be successfully reduced to 9 cubic yards. 12 cubic yards of peat *in situ* will give 1 ton of anhydrous peat.

In the case of large depressions which require draining, and upon which there is likely to be considerable advantages from a large outlay, a sump in



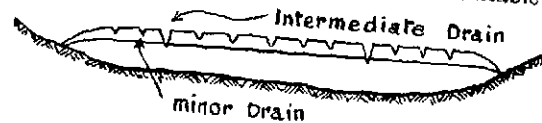
PLAN
FIG. 551.

the centre may be sunk with all the drains delivering into it. The water will then be led off by means of a suitable pipe, sewer, or culvert in tunnel to the nearest watercourse. In case, however, of a suitable outfall being a great



SECTION OF MINOR DRAIN
FIG. 552.

distance away, expense would prevent such a procedure. In limestone districts, however, we will often find faults in the rock in the form of *swallow-holes* (sometimes called *pot-holes*). If these are in a suitable position, they



SECTION OF CROSS DRAINS
FIG. 553.

will form convenient outfalls. It is very necessary, when once open drains have been formed, that attention is paid to keeping them clear.

Now, the selection of pipes for land draining requires a little consideration, although their composition does not naturally get much. Too much sand makes the tiles porous. On the other hand, too much clay will cause them to get out of shape in burning. The presence of lime is a distinct disadvantage. They should always be specified to be "of suitable material, well made, and

burned by skilled workmen, the product when finished being straight and smooth, especially the interior, which must be of uniform bore. When struck together they should give a hard metallic ring and break with a clean fracture."

Regarding the cost of such pipes, which are now practically universally used, the following table will be a help when estimating:—

Diameter of Pipe.	Northumber-land.	Lancashire.	Gloucester.	Banffshire.	Weight per 1000 Pipes.
	£ s. d.	£ s. d.	£ s. d.	£ s. d.	Tons. Cwts.
2	1 1 0	1 2 0	1 5 0	1 9 0	1 1
3	1 15 0	2 5 0	2 5 0	2 10 0	1 16
4	2 5 0	3 5 0	3 0 0	3 15 0	2 7
5	...	4 5 0	4 5 0	5 5 0	3 5

The size of the pipes used will depend upon—

1. The soil and subsoil. Porous soils disposing of their water quickly want the largest drains.

2. The gradient. Naturally a stiff gradient will cause a pipe of given diameter to pass off more water than a flat one.

3. The rainfall. Pipes must be calculated on the maximum rainfall, not the average. In Britain 1 in. of rain in a day is a heavy fall. It is important not to let drains at the end of a stiff gradient get too much charged with water. The "head" under which this will accumulate will tend to burst them; hence the lower pipes in a line of drain should be of a larger diameter, especially in mains having a good rate of fall.

4. The depth of the drains. The deeper the drains are the smaller the diameter required, because the amount of soil above tends to equalise the discharge of rainfall.

The distance apart and the fact of water coming down from higher lands are, of course, important factors.

It may be said that 2-in. and sometimes 2½-in. drains are the most used for minor drains, and 4-in. and 6-in. for mains; for, given a reasonable fall, a 4-in. pipe should easily remove the water of 5 acres of retentive soil or 4 acres of porous, under ordinary conditions of rainfall. It may here be stated that a decided advantage will result when outfalls into open ditches and streams are terminated in a 3-ft. length of cast-iron pipe, having a flap valve thereon, hanging preferably on bronze bolts and nuts. It prevents them silting up and the ingress of vermin, because all drains are liable to chokeage, and to avoid that very careful laying is most essential. A peculiar form of obstruction is that by iron compounds.

This will be especially the case with peaty waters, and is due to iron being dissolved as ferrous carbonate. These, due to aeration in the drains, tend to become oxidized into ferrous oxide and deposit. They occur as a thick reddish-brown substance, principally near the outlet. To prevent such

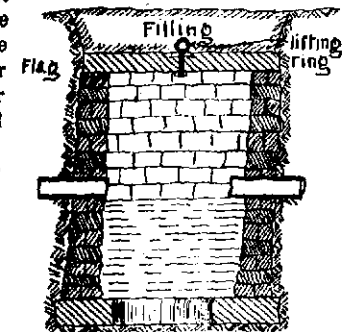


FIG. 554.

chokage, the presence of masonry cesspools (or sumps) on a long line of pipes is advantageous. A form is shown by fig. 554. They would be about 2 ft. square, and should be periodically examined and cleaned.

We will now conclude our remarks with a few examples of cost.

For instance, in an acre of stiff clay we would probably have drains laid 15 ft. apart and 2 in. to 6 in. deep. This would be equal to 968 yds. of drain. The cutting and filling per chain would be 1s. 8d. = £3, 13s. 4d. over the whole acre. The number of 12-in. pipes actually required would be 2905. These at 30s. per 1000 would cost £4, 7s. 2d., making a total cost for the whole acre of £8, 10s. 6d.

Description of Soil.	Depth in Feet.	Distance apart in Feet.	Cutting and Filling per Acre.	Number of 12-in. Pipes.	Cost of Pipes.	Total Cost.
			£ s. d.		£ s. d.	£ s. d.
Clay loam	3.0	22	2 10 0	1980	2 19 5	5 9 6
Friable loam	3.3	30	2 4 0	1452	2 3 6	4 7 7
Sandy loam	4.0	39	1 19 8	1117	1 13 6	3 3 2
Sandy soil	4.0	45	1 14 5	975	1 9 4	3 8 9
Gravel	4.6	60	2 4 0	726	1 1 10	3 5 9

Another way of estimating is as follows :—

The rod in question = $5\frac{1}{2}$ yds.

3-ft. drains. 2-in. pipes.	Easy cutting loam	4½d.-5d. per rod.
	Strong loam	5½d.-7d. "
	Clay	7d.-8d. "
	Strong clay (stony)	8d.-9½d. "
	Very stony clay	9½d.-11d. "

to which add, if the drains are $3\frac{1}{2}$ ft. deep, 1d.-1½d. per rod more; and 4 ft. deep, add 2d.-2½d. more; or if they are only $2\frac{1}{2}$ ft. deep, subtract 1½d.-2d. per rod, and for 2 ft. deep, subtract 2½d.

Other estimates are—

3-in. drains laid 3 ft. deep and 22 ft. apart would cost £5, 2s. 6d. per acre.

This would also include 4-in. pipes for mains.

But if the same conditions held good, and the drains were laid $4\frac{1}{2}$ ft. deep and 45 ft. apart, £3, 19s. 2d. per acre would be a reasonable figure.

To drain peat moss, forming its own channel with drains 3 ft. 6 in. deep and 18 ft. apart, the cost would be £1, 15s. 6d. per acre.

If stone drains were decided upon as being efficient for the purpose, the cost would be as follows per acre :—

Drains 36 ft. apart and 3 ft. deep.	£ s. d.
Opening trenches	1 12 1
Preparing stones	1 3 4
Carting stones	1 6 3
Unloading stones	0 4 5
Filling	0 1 6
Extra for mains	0 10 0
	4 17 7

Some Notes on Irrigation.

The practice of irrigation is practically foreign to British engineers, owing doubtless to our fairly abundant water supply. We say "fairly," because we wish to infer that there are places where irrigation could be carried out with advantage even in our own islands in connection with land drainage schemes, etc. Of course there is a form of irrigation, broad irrigation, in use on sewage farms, but we leave a description to the chapter on Sewage Disposal. In the first place, a knowledge of agricultural chemistry is almost essential. We cannot, however, touch upon the subject here. But we can say that the water from forests and peat morasses are not usually considered suitable for the purpose, especially those which contain a large percentage of FeO_2 , but water from all fertile and arable lands may be used, and any calcareous matter in the water will usually prove a great advantage. Regarding soils, most soils will consist of sand, lime, and clay. Yellow clay is formed from granite, the felspar having changed while the mica has not, the quartz breaking up and forming the sand. Blue clay has no mica, being formed of syenite and greenstone. Sandstone is a silicate, having an alkaline base. Now protoxide of iron will absorb much oxygen, and that is what forms FeO_2 . The former soil is black while the latter is red, the black becoming red when it is turned up and exposed to the atmosphere. Clay in all soils is the fertilising element.

The essentials of irrigation works are—

1. A copious supply of water (the usual requirements for irrigation are .008 = .013 cubic feet per second over each acre treated).

2. An adequate discharge.

3. A regular current all over the land.

Two main systems are in vogue, "bedwork" and "catchwork." *Bedwork* consists of making a series of beds and ridges at right angles to the main channel and along the declivity.

Catchwork consists of a system of distributing the water from one field to another. The former system is, at the same time, the most expensive and efficient, because in the latter system a good many of the fields will get their water second-hand, so to speak. Its use, in fact, is mainly confined to steep land, the main ditches being across the declivity. In bedwork it is usual to make the channels 20 ft. wide at the junctions, tapering to 12 ft. wide at the ends. They have a fall usually of 1 in 500, and are practically limited to 700 yds. in length.

The subsidiary conducting channels vary in size 15 ft. wide by 3 ft. deep, being sufficient for 300 acres. The distance between feeders and drains would be 4-5 ft. Branch conductors are known as *floating troughs*. Their usual fall is very flat, 1-1500 to 1-3500 being sufficient. The remarks on overfalls will also apply to them, so get such flat gradients. The radius at changes of direction must be at least 100-150 yds. All aquatic plants must be cut down as much as possible.

For suitable measuring weir, see chapter on Hydraulics (figs. 524 and 525).

Drainage of Low-lying Lands.—Another important aspect of land drainage is the draining of low-lying lands near the sea-coast, often termed *sloblands*, and generally consisting of land previously reclaimed from the sea by an artificial embankment. Sometimes the land is high enough to allow of gravitational drainage to the sea, but very often steam pumping has to be

"The best mode of draining a district of this sort is by means of a canal extending completely through it, which acts alternatively as a reservoir and a channel. The *top water-level* of the canal is to be fixed so as to give sufficient declivity to the branch drains. Its *low water-level* will be above that of low water of neap tides to the extent of 1/15 part of the rise of such tides. The space contained in the canal between those levels is the *reservoir room*."

When levels have been taken and the best position for discharge decided upon, we then lay down the proposed lines of drains on the plan. It is then necessary to find the required size of each, and the necessary amount of earth-works, etc., upon which to base an estimate.

These items should be tabulated as follows:—

1. Area of land.
2. Greatest length.
3. " width.
4. Rainfall.
5. Discharge period per day (hours).
6. Quantity of water due to rainfall in cub. ft. per second.
7. Number of main drains.
8. Distance apart proposed for subsidiary drains.
9. " " minor "
10. Area drained by each minor drain in acres and decimals.
11. Size of main.
12. Fall " (ft. per mile).
13. H.M.D. of main.
14. Discharging capacity, using Equation 293. (This may be reduced at its middle length to take only half the quantity by altering the size given in No. 11.)
15. Number of subsidiary drains.
16. Fall " "
17. Dimensions of subsidiary drains, allowing continual discharge and using Equation 293, and considering the rainfall supplied by each of the minor drains.
18. Size of minor drains.
19. Area of land occupied by actual drains.
20. *Sluices*—number of; width of; depth of water on; velocity through, ft. per second.

CHAPTER XXIV.

PUMPING MACHINERY.

At first sight, perhaps, a chapter on pumping engines may seem to be out of place in a book of this sort, and more a matter for mechanical engineers. A little consideration will, however, soon dispel that idea, because the modern municipal engineer will in nine cases out of ten have pumps under his direct charge to oversee, test, and possibly to advise extensions.

Perhaps the simplest automatic pump, or rather a machine for lifting water, is the hydraulic ram. It is illustrated in fig. 555. Its object is to raise

water to a considerable height by using the potential energy of water at a smaller height. At A is attached the supply pipe, called the drive pipe, 10 to 20 ft. long; it is connected with the supply tank. The bell F contains air to act as a cushion. There is a valve B, having a weight keeping the valve open normally. Suppose water starts to flow out of this valve, it acquires velocity, increasing, too, so much as to overcome the weight and close the valve. Rushing onward, it gains considerable pressure, so much so that the ball valve lifts and the water flows up the supply pipe. Now the valve B falls again and the process is repeated. The action of the hydraulic ram illustrates the conversion of potential energy into kinetic energy, kinetic into pressure, and pressure, again, to potential (*vide* Hydraulics). Now, referring to the line diagram fig. 556, we have H = delivery head and h = supply head; then the efficiency of the apparatus

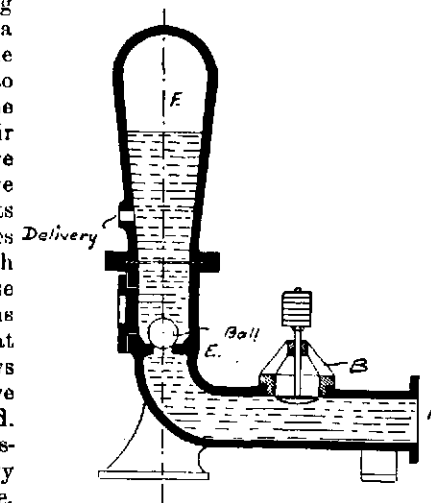


FIG. 555.

$$= \eta = 1.12 - 0.2 \sqrt{\frac{H}{h}} \quad (345)$$

The quantity of water which will run to waste by way of the dash valve B

$$= \frac{qH}{\eta h} \quad (346)$$

η being as in Equation 345.

resorted to. Again, on a gravitational scheme the land may be so low or the floods so abnormal that a special pumping plant has to be put down to assist in removing the flood waters from the lower levels. In any scheme of this kind it is usual to construct catchwater drains the whole way round the area concerned, so as to get rid of the water from the uplands once for all and lead it to a separate outfall. This drain is termed a *marsh fence*, and if the low land happens to be traversed by a *natural* watercourse it may deliver into it if the water likely to come down will not overcharge it. The drains of such low-lying lands are surface drains of large dimensions, as shown in fig. 541, and must be carefully calculated in respect to their discharging capacity from existing rainfall records. It is well to bear in mind in such calculations that an abnormal rainfall does not necessarily cause floods in low lands; on the other hand, it is long-continued rainfalls which do the damage, and heavy rains coming on after continued bad weather. Various amounts of rainfall can be considered when calculating the maximum discharge required. To calculate on the maximum would be unnecessary, owing to the usual short duration and unlikely occurrence of such falls. Consequently a mean must be arrived at, and in such a case we cannot do better than follow the practice of the early engineers in this direction, who usually calculated on a continuous fall of a quarter of an inch of rainfall in twenty-four hours, making no deductions for any losses by evaporation or otherwise. All springs should be treated as before advised, and outfalls provided for them. The next question is the level of the sill of the sluices, which will be the starting-point of the whole scheme, because upon the level of this point will depend the capacity and duration of discharge of the waters, together with the amount of ebb tide and the depth of water on the sluice sill. It is the best practice where possible to aim at a continual discharge of six hours, viz. three hours on the ebb and three hours on the flood tide, which will, under ordinary circumstances, necessitate the sill of the sluice being 2 ft. above the lowest spring-tide level. Even then, if there is a wind backing up the tide, this will be reduced, but, on the other hand, a good pressure at the back from the drainage area will increase it. Again, the minimum level of the sluice sill below the general level of the land must not be less than 2 ft., because 18 in. is the minimum depth allowable between the surface of water in the drains and the land, while allowing always 6 in. of water on the sluice sill. Hence we find that we must provide for 4 ft. fall between the land and the lowest spring-tide level before we can start the design, and the amount stated is little enough.

The water in the drains should never stand higher than 18 in. below the surface-level if the land is to be properly dry, and 2 ft. is advisable. In heavy clay, 2 ft. 6 in. would be better, and 3 ft. is quite the minimum in friable loam, because it is essential that the under-drains should run clear, and always have their ends above water-level. Hence we see that, with the levels generally at the engineer's disposal, in such works great accuracy is required in setting out the works, hence the form of channel demands attention, so that the greatest discharging capacity with the least fall is obtained, and the reader should refer to the question under Hydraulics. At the same time the form of channel referred to therein of a depth of 1 to a width of 2 is inadvisable in practical land drains, which usually have very flat slopes. A well-constructed main drain will flow efficiently with a fall of 3 in. per mile if properly laid out, and flat slopes do not appear to be any real impediment to the flow; on the other hand, such a form of drain is of the utmost service

in dealing with floods, inasmuch as the increasing width of section at the top provides a reservoir for flood water, which is a great consideration in pumping schemes. The slope of the sides will, of course, vary according to the soil through which they are cut. They must be *flatter* than the natural slope; and where a stiff soil tops a lighter one and the drain extends down to the latter, the sides have very often to be pitched or otherwise treated to prevent them falling in, no matter what slope they have. Besides the main and subsidiary drains and the catchwater drains referred to, in reclaimed land there is generally a drain on the landward side of the bank and at the base of it. Its purpose is to intercept any leakage through the bank and to deal with water thrown over in very severe storms, although the bank should be so designed that neither (especially the latter) should occur, but this ditch is usually kept about 20 ft. away from the actual foot of the slope, and is about 10 ft. wide at the base of excavation. The reason for removing it back is because, if placed at the actual foot of the bank, it would be likely to induce percolation, or cause it to slip, and the latter contingency is reduced to a minimum by using the 20 ft. referred to as a roadway. The drain itself is called a *delph*. The excavation also makes material for the bank. The depth of water allowed in the delph is usually 3 to 4 ft., and the whole depth 5 to 6 ft., but in sandy soil only a very small drain is allowed because of the liability of bank and drain to slipping. The rule given by Mr Beazley (in his treatise on the reclamation of land) for the distance between bank and delph is—"care should be taken that the minimum breadth of undisturbed surface is such that the inner slope of the bank if produced would nowhere intersect the nearest side of the excavation forming the delph—and even this as a minimum in cases only where it is not possible to give a greater breadth." The sluices at the end of the drain are usually made of heavy masonry. The doors are self-acting, and usually of the canal lock-gate type, meeting together when closed at an angle of 130°. There is generally a bridge over the sluice, which may have one or more openings according to circumstances. The piers and abutments are generally extended on the seaward side a distance from two to three times the width of a single opening. They have pointed cutwaters and grooves for draw-doors, which are raised by chains and rack-and-pinion motions, or may be stony sluices, of the form in fig. 506A. It is these which regulate the height of water in the drains. Small sluices have doors of the flap-valve type, but are very often of elaborate construction, so as to respond to the slightest back pressure. By reason of the nature of the land on which these works are executed, the foundations (often on alluvial deposits) are frequently most difficult, and piling, and in sand sheet piling, are generally resorted to. The site of the sluices is also an important factor because of the liability of sand to drift and silt up the outfall, and training-walls have often to be erected along the channel from sluice to low water, especially if the foreland* is flat and marshy. It is not necessary that sluices should always discharge into the sea. They often, in fact, discharge into a river.

The site, however, is again important, and the discharge should take place on a concave bend. The reason for this is that there is always deep water at these points, and silting up is less likely, especially when the river is low. The outflow from the sluice must be parallel or nearly so to the ebb-tide current.

Professor Rankine, in his *Civil Engineering*, pp. 727 and 728, says in reference to draining low lands:—

* The foreland is that stretch of ground between the foot of the bank and the low-water mark of ordinary spring tides.

The actual water delivered per stroke

$$= \frac{Qnh}{H} \quad (347)$$

The correct diameter of the drive pipe

$$= D = 2\sqrt{Q+q} \quad (348)$$

The correct diameter of the supply pipe

$$= d = \frac{D}{2} \quad (349)$$

The length of drive pipe required in feet

$$L = H + 1.2\left(\frac{H}{h}\right) \quad (350)$$

while the content of the air vessels should be equal to $0.055d^2 H$. The diameters D and d are in ins. (Molesworth's Tables) (351)
The fall may be anything from 10 ft. to 18 in., but above 10 ft. frictional

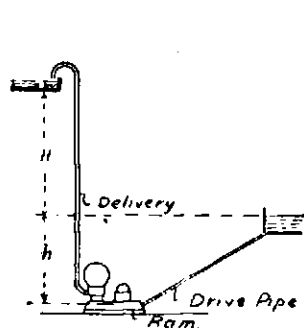


FIG. 556.

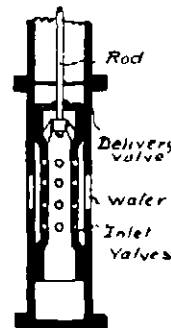


FIG. 557.

effects cause jar and wear to become very excessive. Rams may be used coupled together also. Hydraulic rams can be well recommended, but their use is not widely taken advantage of. They are very simple, rarely get out of order, are cheap, and work months on end in out-of-the-way places without attention. But they must in the first instance be well designed and fitted. All valves should be of gun-metal, all joints faced, and a snifting valve provided to supply the air which is of necessity absorbed by the water. In fixing, they should be placed below ground to avoid the effects of frost, and a strainer must in all cases be provided. Regarding actual pumps, they may be divided up as—

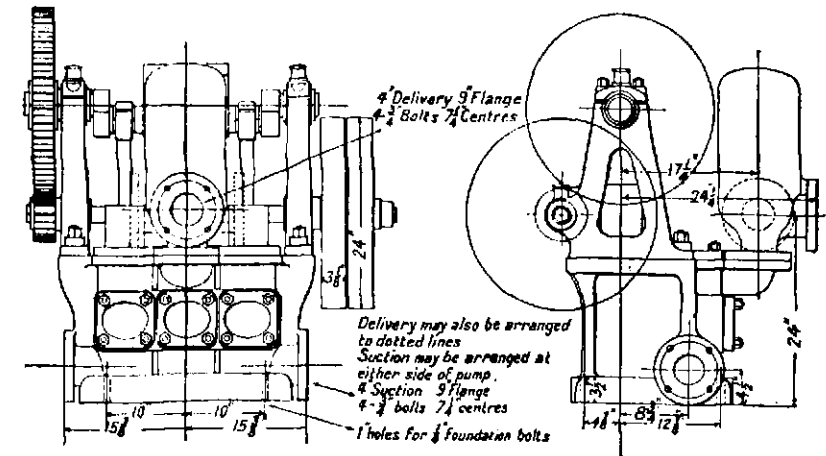
1. Lift pumps. 2. Plunger or force pumps. 3. Centrifugal pumps.

In a lift pump the water is drawn through the suction pipe on the upstroke of the bucket, forced through the bucket valve as the pump descends, and lifted again on the upstroke. A diagram of a bucket pump appears in fig. 557. It is about the only form of really efficient bucket pump, and depends for it principally upon the arrangement of small inlet valves.

Plunger pumps are pumps in which water is drawn through a suction pipe, and is displaced by the action of a plunger or piston which forces it through the delivery valve.

The principal form of plunger pump which concerns the engineer is the three-throw pump, or, as it is usually termed, the treble ram pump. This arrangement will prevent the flow of water being too intermittent, as would be the case of only one plunger.

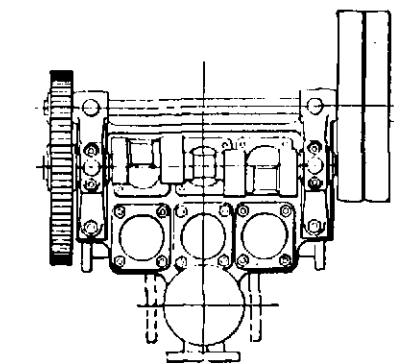
Centrifugal pumps have a fan or impeller, which revolves usually at a high speed inside the casing in which it is placed. It creates a vacuum which will



suck up the water. It is most important that the engineer who is entrusted with the work of selecting a pump makes careful inquiries respecting that pump from the manufacturers. When ordering it he must state—

1. The purpose of the pump.
2. The maximum quantity to be pumped in gallons per hour (this must be guaranteed, together with the fuel consumption).
3. The total head (suction plus delivery).
4. The amount of suction.

Where fuel is expensive, the distance to be pumped great, and the water clean, there is nothing to beat the vertical triple-expansion pump for efficiency. For similar purposes but for smaller duties the treble ram pump,



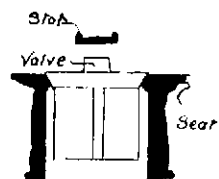
FIGS. 558, 559, and 560.

driven either by belt, electric motor, or gas or steam engine, by means of gearing, is next in favour for waterworks pumping of all kinds. Then, for sewage, irrigation, emptying docks, and in fact moving any large volumes of water to only a moderate height, the centrifugal pump has given great satisfaction in the past, and continues to find favour; and now, with the introduction of multi-stage pumps, great heads can be economically obtained with these pumps too. They are practically always

direct driven by the engine or motor, belts being only used for temporary purposes.

Some of the most common types of pumps will now be described, with illustrations.

First of all we shall describe the treble ram pump. By the courtesy of Messrs Frank Penn & Co. of Manchester, figs. 558, 559, and 560, of a vertical pump for belt-driving by a gas or steam engine, are presented. The drawings are practically self-explanatory. The pump barrels are of close-grained cast iron, and, together with the valve chambers and vacuum vessel, are formed in one, having a trough which collects leakage from the glands. The valves and seats are of gun-metal and of the mitre type, fig. 561. The rams are truck type, and work through neck rings and cast-iron glands. The crank-shaft is of forged steel, likewise the connecting rods. They



Plan looking Up

FIG. 561.

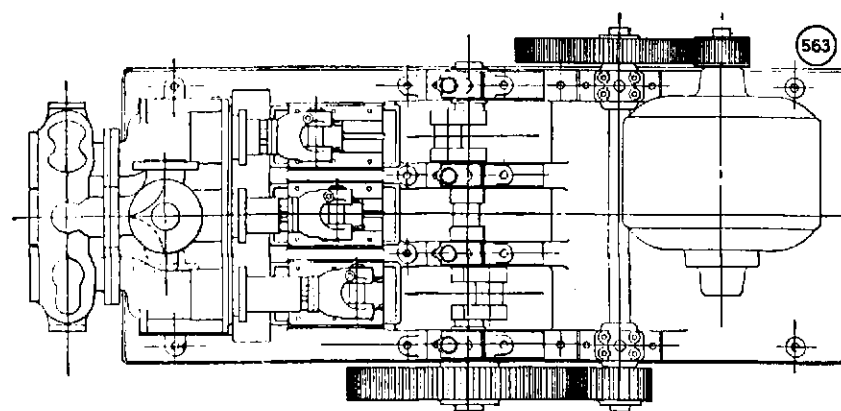
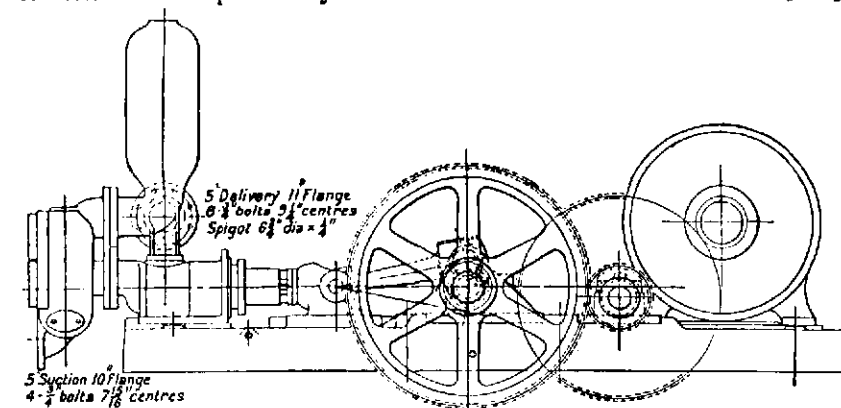
run in brass bushes of the usual type. The air vessel is of cast iron. The pump is suitable for a head of 180 ft. Another very favourite type of pump is the horizontal treble ram pump. A very good form by the same firm is shown in figs. 562 and 563, driven by electric motor of the continuous current type. The details are much the same as the descriptions just given, except we have the gearing, which is of cast iron. A raw-hide pinion should in all cases of electric driving be provided on the motor shaft; it reduces the noise, and wear and tear also. They are useful on any gearing. Such pumps are successfully driven in the same way by gas engines, by means of a pinion of raw hide likewise. The only addition is a friction clutch to allow of the engine being started

by hand and getting up speed. For waterworks pumping on a larger scale than for which an ordinary three-throw pump would be suitable, the direct-driven plunger type pump comes next in order, especially for dealing with large continuous volumes of water. A fair example of what has been done with these sort of pumps is the installation at the Ilkeston and Heanor Waterworks. Here there are three sets of vertical triple expansion condensing engines. By the kind permission of Messrs Tangyes of Cornwall Works, Birmingham, who were the builders, it has been possible to reproduce a plan and elevation of the pumping plant in figs. 564 and 565.

The cylinders of the engines are 15 in., 24 in., and 40 in. respectively, in diameter. The stroke is 33 in., while the pumps are of the direct-driven plunger type, 10½ in. in diameter × 33 in. stroke. These air pumps are worked off the main engines, which is usual in such cases, and the steam supplied by Lancashire boilers. Regarding boilers, we have not much to say regarding their respective merits here. Suffice to say, that for large continuous volumes of steam the Lancashire and Cornish types having one or two internal flues, as the case may be, remain still unsurpassed, provided the pressures are moderate, say up to 150 lbs. per sq. in., but for intermittent working, sudden overloads, rapid steam generation, and high pressures, the water-tube boilers of the Babcock or Hornsby type are rapidly superseding their use.

Perhaps one of the best-known types of pumping engines for waterworks is the Davey engine. For steam consumption it certainly holds its own any-

where. A good example is at work at the Walthamstow pumping station of the London Waterworks, and is of the horizontal type, and is fully described in Messrs Tudsbery and Brightmore's Treatise upon Waterworks. A more recent type is those supplied to the Rand Water Board, manufactured by Messrs Hatherly, Davey & Company, Leeds. In 1909 they held the world's record for steam consumption. By the kindness of the manufacturers the pumps



FIGS. 562 and 563.

are illustrated in figs. 566 and 567. The following is a description of the pumping plant as erected at Zwaartkoppes:—

The scheme for water supply now approaching completion includes a number of shafts and boreholes in the valley of the Klip river, from which water is drawn. The pumps at these shafts are all actuated by electric power, distributed from the central station, in which the main pumping plant for delivering the water to the Rand, and which forms the subject of this description, is situated.

The difference of level between the central pumping station and the summit level at Turfontein Nek is 810 ft. This is increased by the head

necessary to overcome the friction of the water in the main, to a degree which varies with the volume which is being pumped. The pumps are capable of delivering against a head of 960 ft. This figure may be approached in future years while pumping the maximum volume of water, say 9·6 million gallons daily, and with the internal surfaces of the main more or less corroded.

Johannesburg is situated on the top of a hill, an exceptional position for a large town, and, owing to this, the height to which the water has to be pumped is altogether exceptional, and unparalleled in the case of any other waterworks installation of considerable size. After fixing on the most suitable site for the main pumping station, it was necessary to consider whether gas engines or steam pumping plant would best fulfil the requirements of the case. At that time—early in 1905—great expectations were entertained of the economy likely to arise from the municipal gas-engine installation. Steam plant, however, was finally decided upon. The engines work on a steam pressure of 180 lbs., with 500° of superheat. Each engine is capable of delivering 84,000 gallons per hour, against a normal head of 900 ft. The maximum guaranteed load was 100,000 gallons to 960 ft. per hour. The guaranteed consumption was 11·5 lbs. per water horse-power hour. The tests for steam consumption were undertaken by Professor Orr of the Transvaal University. The chief engineer to the Water Board is Mr D. C. Leitch, M.I.C.E., and the results of the test are as occur below.

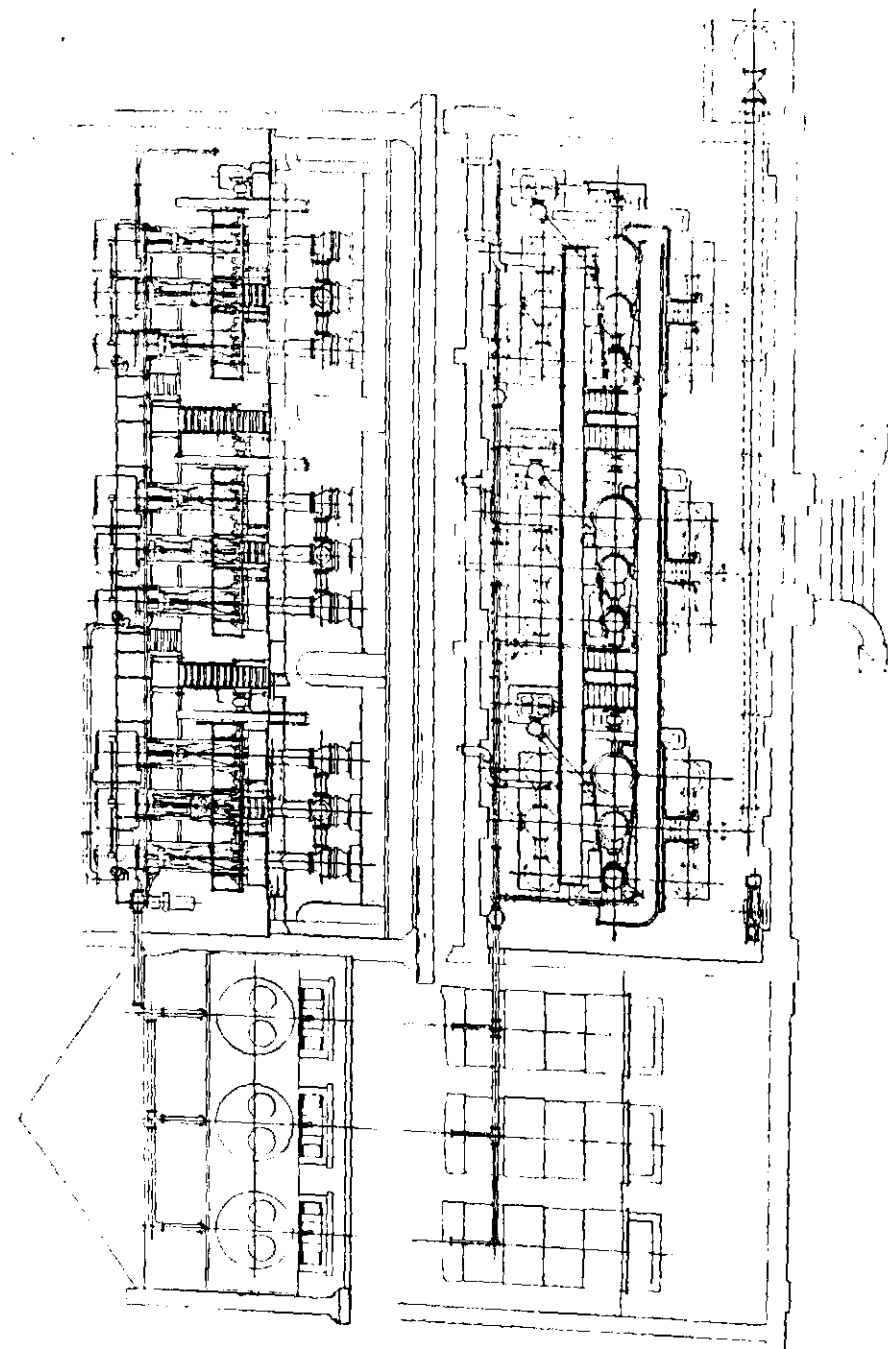
The size of the steam cylinders are 23 in., 43 in., and 64 in. respectively, in diameter. The stroke is 3 ft. The valves are of the Corliss type, worked by gearing. The special feature common to Davey pumping engines is the rocking discs, connected to the engine and pump connecting rods respectively, by means of which a special mechanical advantage is gained, because while the engine is making its stroke the pump piston or bucket is varying in velocity relatively to that of the engine piston, the ratio of the variation being determined by the relative positions of the point of attachment on the discs of the engine and pump pins respectively.

ZWAARTKOPJES ENGINES—PRINCIPAL RESULTS.

	Engine No. 5.	Engine No. 4.	Engine No. 3.	Engine No. 2.
Average efficiency of engine	95·7	95·4	97·37	97·72
Average efficiency of pump	93·8	97·0	94·66	95·59
Combined efficiency	89·77	92·58	92·17	93·41
Million ft.-lbs. per 1000 lbs. of steam	171·33	174·73	170·80	183·61
Steam pressure	180·8	182·25	180·66	183·6
Vacuum	23 in.	22·6 in.	22·6 in.	22·6
Barometer	25·3 in.	25·4 in.	25·0 in.	25·6 in.
Steam temperature	499·9	499·4	500·3	499·7
Slip	3·92	1·22	3·58	2·54

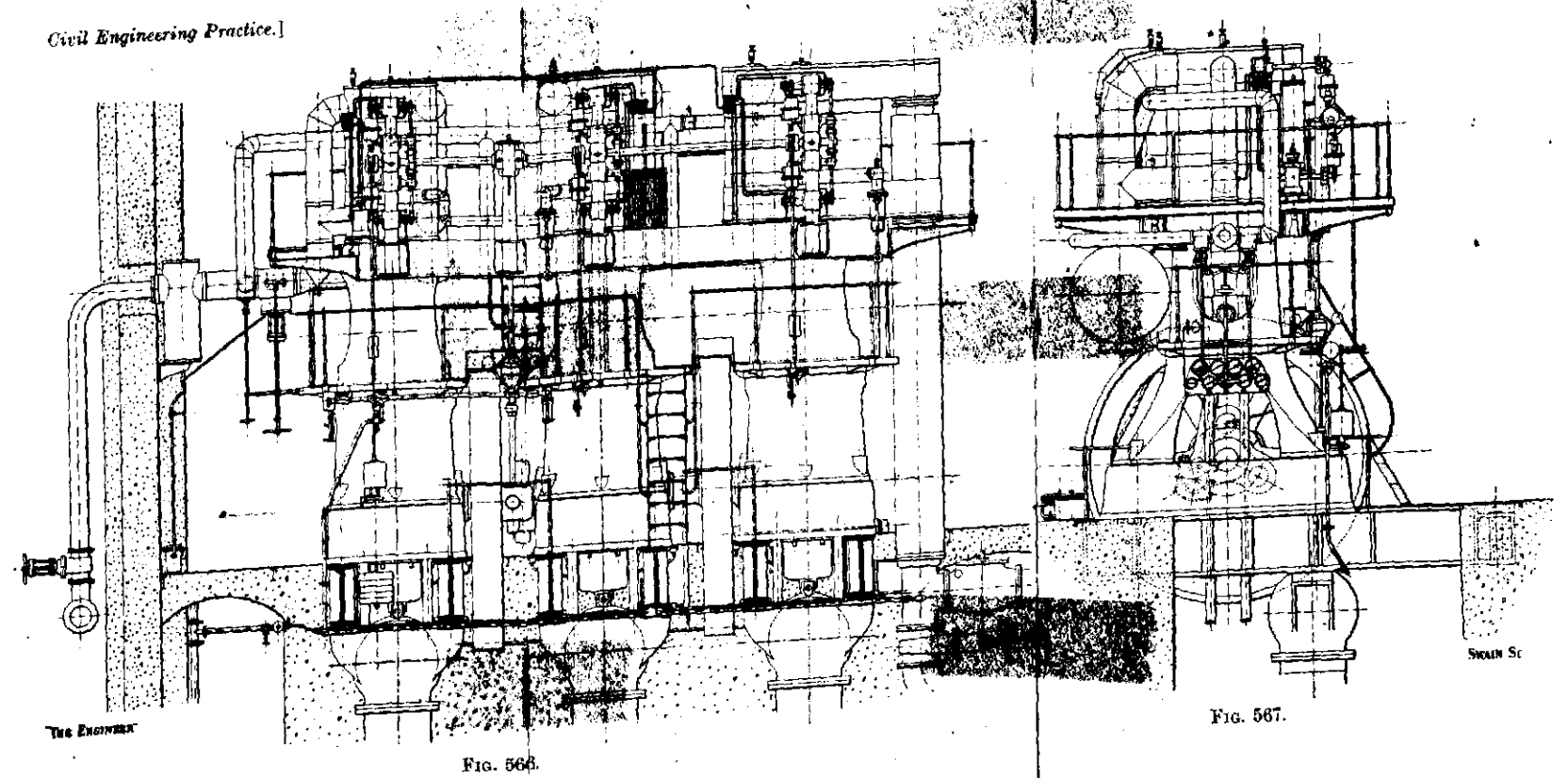
All losses taken into account and figures based on work done in actual delivery of water. The suction water circulates through the condensers, and the engines drive the air pump as well as a supplementary pump to raise the air-pump discharge to the hot-well tank.

398A



Figs. 564 and 565.

Civil Engineering Practice.]



{To face p. 398.

delivery of water. The suction water circulates through the condenser, the engines drive the air pump as well as a supplementary pump to raise the air-pump discharge to the hot-well tank.

Following are fuller details of the tests on the Rand Water Board engines.

REPORT OF TEST OF NO. 2 ENGINE BY PROFESSOR ORR.

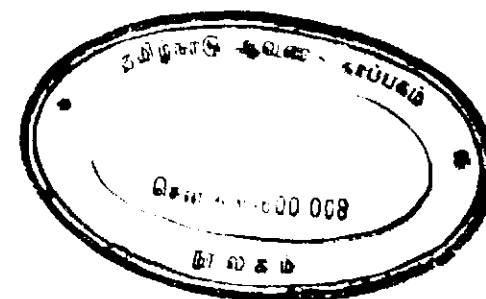
Summary of Principal Results—Engine No. 2.

Steam cylinders: H.P. = 23 in.; I.P. = 43 in.; L.P. = 64 in. by 3-ft. stroke.

	Test No. 1. Normal Load. 1908.	Test No. 2. Overload. 1908.
Date of test	May 30th	May 31st.
Test began	10 a.m.	6.30 a.m.
Test ended	8 p.m.	4.30 p.m.
Duration of test	10 hours	10 hours.
Barometer, ins. of mercury	25.3	25.3
Pressure of steam at engine stop-valve— lbs. per sq. in.	183.6	183.7
Vacuum (exhaust steam just entering con- denser), ins. of mercury	22.66	22.6
Temperature of steam at engine stop valve, degrees Fahr.	499.5	500
Revolutions per hour	2028.6	2475.1
“ “ minute	33.81	41.25
Piston speed, ft. per minute	202.9	247.5
Pump displacement per revolution—cub. ft. “ “ “ (slip neglected), lbs.	7.0686	7.0686
Water delivered per revolution, lbs.	440.35	440.49
Water pumped per hour (slip neglected), lbs.	428.73	429.69
Water pumped per hour (slip neglected), lbs. “ “ “ (actually delivered in Turfontein reservoir), lbs.	893.294	1,090.257
Water pumped per hour (slip neglected), gals. “ “ “ (actually delivered in Turfontein reservoir), gals.	869.722	1,063.526
Slip of pump, per cent.	89.329	109.026
Effective head of water pumped, ft.	86.972	106.353
Horse-power, water delivered (slip neglected) “ “ “ (actual)	2.64	2.45
Water collected from air-pump discharge per hour, lbs.	929.7	975.5
Water collected from jacket and reheater drains per hour, lbs.	419.44	537.14
	408.37	523.97
	3990.3	5080.8
	462.4	568.8

A pumping engine on somewhat the same principle as the Davey engine is illustrated in figs. 568 and 569. It is one of the engines of the Sutton-in-Ashfield Waterworks, and affords a very good example of a double stand-pipe pump placed in a borehole. As before stated, the height of suction for a pump is limited to 30 ft. theoretically, but in practice 25 ft. is rarely exceeded, and in all such cases it is absolutely necessary to have a retaining-valve at the foot of the suction pipe. The engine actuating the pumps is of the horizontal girder type, having cylinders worked by slide valves 26 in. in diameter by 40-in. stroke, while the pumps are 14 in. in diameter by 40-in. stroke.

The rocking discs are placed, together with the bearings and other details,



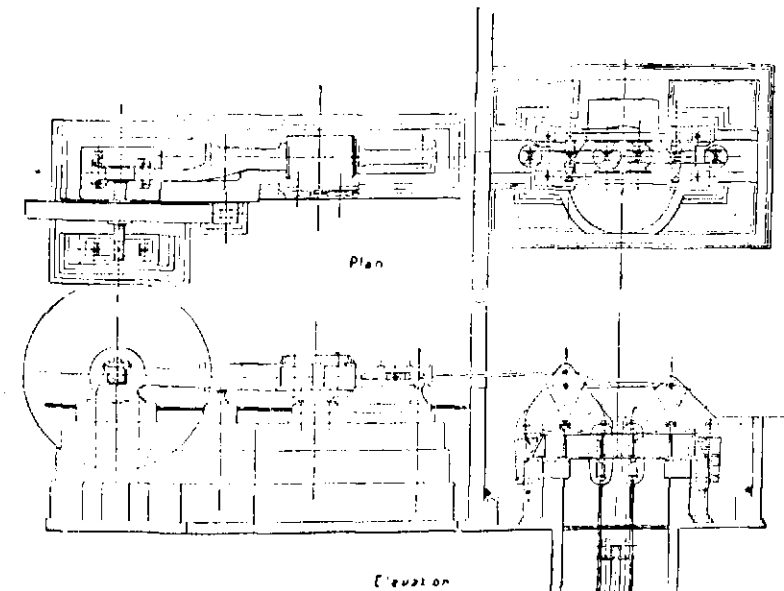


FIG. 568.

on heavy cast-iron girders spanning the well. The pump rods work on guides placed at intervals in the well. The discs are in turn worked by tail rods from the main cylinders, and are suitably balanced. Altogether, the set may be said to be very compact and well-designed. The work was executed in 1897.

Modern waterworks pumping practice tends to favour that type of pump which is entirely separated from the engine, viz. rod pumps, the pump being placed in the well or borehole as illustrated in figs. 568 and 569. The choice of the well or borehole depends upon certain conditions of water-level, the depth of which will vary with the pumping, and also as to whether the strata will yield water freely. The matter is fully discussed under Waterworks. Borehole pumps are invariably bucket pumps, whereas well pumps may be of the ram type up to within 25 ft. of the low-water level. The former type of pump can, however, be sunk to any convenient depth, 300 ft., however, being considered by engineers to be a practical limit. The advantage of the ram pump lies in the fact that if the area of the valve passages be large it may run fast. This, however, is not the case with bucket pumps, in which it is essential to avoid valve slip,

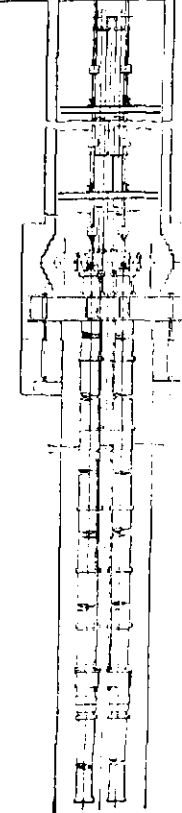
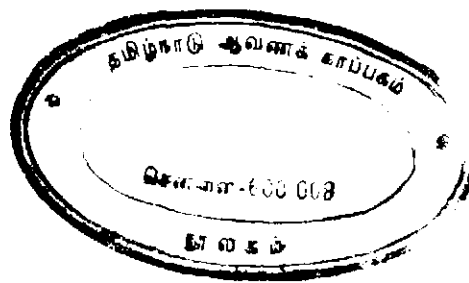


FIG. 569.



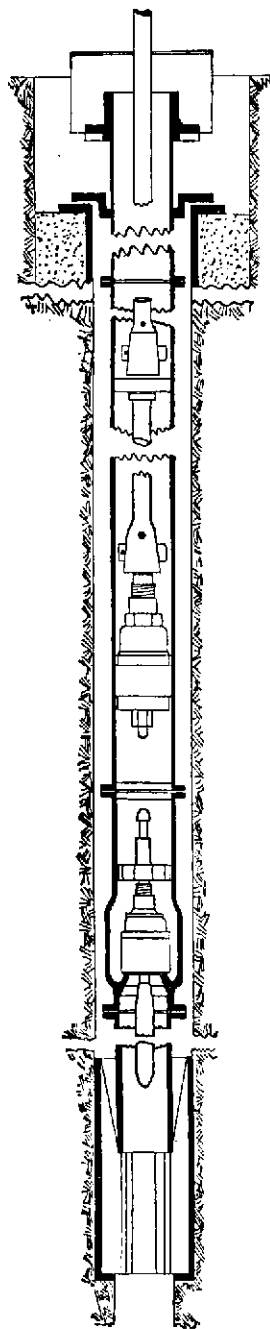


Fig. 569A.

which primarily depends on the tightness and sensitiveness of those valves. Considering the actual construction of these pumps, the use of unpacked pistons is not to be advised; however accurate the work may be when new, it soon will get out of truth. The design of the rod joints needs more attention than is usually bestowed on them. They must be very firm, yet easily disconnected, points in favour of the jib and cotter principle; they must be loaded to descend by their own weight and be well guided. Reference to figs. 568 and 569 will show the weights provided for this purpose. These pumps are usually designed for a long stroke, the reason being that most wear of the working parts takes place at the reversals of stroke. Rarely, if ever, do we see the pumping engine placed over the borehole, for obvious reasons, in which case we have to resort to either gearing or the use of connecting rods. With gas engines running at high speeds gearing is essential, but with steam engines, which for this purpose run at specially low speeds owing to their special design to give very high efficiencies, coupling rods are used. In figs. 566, and 567 we show a system for use where floor space is limited, and a vertical engine necessary. Figs. 568 and 569, however, are much less costly, but the engine which is now much favoured is the non-rotative engine of the original "Davey" type. The only disadvantage is the unavoidable variable length of stroke. This, however, is of minor importance, because the result of observations over long periods of working show that a stroke of normal length will result in one of average length. Another disadvantage in this type of engine is the large cylinder clearance necessary. On the other hand, the pause at the end of each stroke is a distinct advantage, and with cheap coal the pump stands unrivalled. The latest example of a successful waterworks pumping plant is the Stocks Well extension of the Widnes Waterworks.

The principal dimensions are as follows:—

- Cylinders, 23, 38, and 60 in. diameter.
- Two borehole bucket pumps, 20 in. diameter.
- Double-acting force pump, 20 in. diameter.
- Stroke of engine and pumps, 6 ft.

The duty required by the engineer (Mr Isaac Carr) was to pump 3,000,000 gallons of water in 24 hours from a depth of 250 ft. to the surface, and thence to the service reservoir at Pex Hill, a distance of 2 miles and 150 ft. above ground-level at Stocks Well. The total lift specified was 500 ft.,

which includes 100 ft. friction head. The steam cylinders are steam jacketed and have Corliss valves placed at the four corners, the valves being actuated by water taken from the pressure main by a patent valve gear designed to give a specially good steam distribution. There is also a differential gear. The borehole pumps are shown in fig. 569A, and are described as follows: The working barrels and buckets are of cast iron. The clacks are fitted with cast-iron valves of the double-beat type, having gutta-percha beats. The clack boxes are of cast iron, secured to working barrels, but of a somewhat large diameter, in order to give increased waterway round the clacks. They are furnished with machined guide ribs to keep the clacks in their proper places. The clacks rest upon flat machined faces, and are provided with steel weights to keep them from lifting during the surface stroke. All risk of their becoming jammed when being withdrawn is obviated. The clack valves are furnished with gutta-percha beats, similar to those of the bucket valves. The top of each clack has an annular guide. The clacks are furnished with fish heads, and an effective fishing tool is provided, together with a galvanised flexible steel wire rope. The lifting tackle is designed to lift 25 tons. The results of the trial are as follows:—

Date of trial	5/11/08
Duration of trial	12 hours
Steam pressure at engine	175.5
" temperature at engine	459° Fahr.
Superheat	81.5° Fahr.
Vacuum in condenser	28.4 in.
Steam pressure in H.P. receiver	44.6 lbs.
" " I.P.	0.76 lb.
Total number of double strokes	12,340
Double strokes per minute	17.138
Indicated H.P. of H.P. cylinder	160.37
" " I.P. "	172.87
" " L.P. "	157.76
Borehole gauge (less 11.16 ft.)	151.56 ft.
Delivery " (plus 13.87 ft.)	347.27 ft.
H.P. of borehole pumps	128.08
" force pump	286.03
" absorbed by gearing	3.45
Total net pump H.P.	410.66
Mechanical eff.	83.63 per cent.
Air-pump discharge	61,611 lbs.
Mean temperature of discharge	76.8° Fahr.
Jacket drains	6405 lbs.
Water supplied to vacuum spoiler	6 lbs.
Total steam used by engine	68,010 lbs.
Jacket steam per cent. of total	9.41 per cent.
Steam used per hour	5667.5
" " I.H.P. per hour	11.42 lbs.
" " " " "	13.8 lbs.
Duty of 1008 lbs. of steam	144,616,000 ft.-lbs.
" 1000 "	143,468,000 ft.-lbs.

Now a type of waterworks pumping engine wherein economy has been brought to a fine art is the Worthington triple-expansion engine, illustrated

in figs. 570, 571, and 572. The cylinders are arranged in tandem fashion; the valves are of the Corliss type, worked by rocking levers from the cross-head. The engine has a condenser and air pumps, etc., placed below the cylinders, worked by rods as shown. Saving of space, quiet working, accessibility, ease of repair, are all points in favour of the Worthington engine, of which so many examples are at work in America at the present day. The Worthington engine, it will be observed, is direct-acting, there being no fly-wheels. At the same time we have a high rate of expansion and a uniform pressure on the pump pistons, which is most essential.

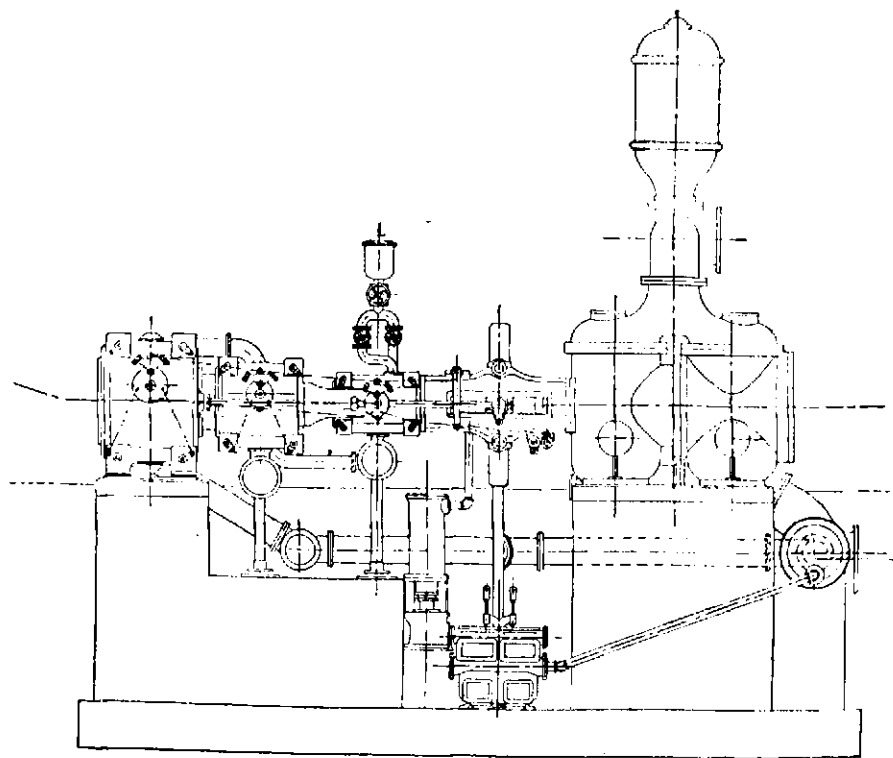


FIG. 570.

It will be seen that there are really two engines side by side, one working the valves of the other, in a very similar way to the ordinary duplex pump for boiler feeding, which, however, is not an efficient machine by any means, and is not suited to waterworks pumping in its simple form with no expansion. The valves on the H.P. cylinders admit steam and deal with the exhaust at the same time. Separate valves, however, are provided in the H.P. cylinders. Now one of the peculiarities of this ingenious piece of mechanism, so typical of American practice, is that the steam cylinders are free to move in the direction of their length. This is to take up the effects of temperature which may be considerable in a large engine, and would perhaps seriously alter the valve motion. There is also a distinct pause at the end of each stroke. This

is an advantage, because it allows the valves to settle and prevent slip. But the crux of the whole invention lies in the special compensating cylinders, which so successfully do away with the fly-wheel. It is arranged in the high- and low-pressure cylinders, that in the first half of the stroke the excess of pressure above the mean is equal to the deficiency below mean pressure in the second part of the stroke. Below the cylinder are two oscillating cylinders. There is one on each side of the piston rod, their trunnions being at right angles to them. They are also attached to the main framing. They have pistons in them, the rods of which are connected to a sleeve on the main piston rod. It is the

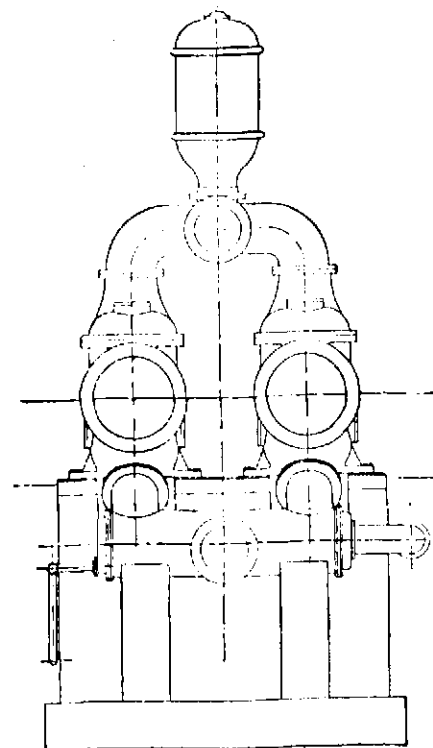


FIG. 571.

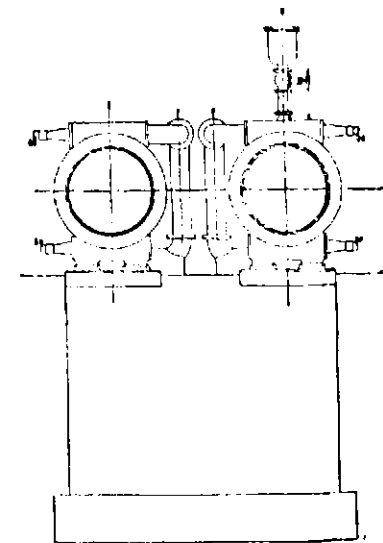


FIG. 572.

duty of these pistons during the first half of the stroke to force water against the pressure of an accumulator. This is given back in the latter part of the stroke, and so compensates the loss of pressure. They are designed of suitable size to fulfil these conditions. It may here be remarked that in all pumping engines for waterworks it is the usual practice to pump the whole or part supply through a surface condenser. We then avoid the circulating pump, all to the advantage of the engine. We now seem to have touched upon most of the rotative and direct-acting pumps. They are the usual types which find favour with waterworks engineers, because the head against which such pumps work is usually considerable. Another type of pump still much used in older installations, but not now built, is the old Cornish engine. It

was formerly used much for mine draining; in fact that seems to have been its original duty. They were of two types—beam engine and direct-acting. In the beam engine, steam was only admitted on one side of the piston, the top side, the weight of the pump rods being sufficient to bring up the piston again. In the direct-acting form, however, steam is admitted below the piston. The peculiarity of the Cornish engine (of which the direct-acting form is sometimes called the Bull engine) is that the cylinders are never open to exhaust. Perhaps this accounts for the still predominant efficiency of the Cornish engine amongst steam pumps. These engines usually work pumps in boreholes or wells by means of long rods. The rods are made of pitch pine, because they are more suited to shocks than steel ones. For ram pumps they are usually from 10 in. to 14 in. square, and for bucket pumps $4\frac{1}{2}$ in. to 6 in. square. The rods are proportioned to the weight of water in motion during the stroke of the pump by means of balance weights. The same reasoning applies to all pumps similar to figs. 568 and 569.

We have now touched lightly upon rotative, non-rotative, and direct-acting pumps as regards their use in waterworks schemes, and similar instances of lifting water to any considerable height. All the drawings should prove of value to the student, showing as they do the best examples of modern practice. On all such pumps large air vessels must be provided to produce a steady and regular flow. The size is arrived at as follows. Its diameter must never be less than that of the plunger; $1\frac{1}{2}$ the size is about the average, and the height is equal to, when P = pressure in lbs. per square inch, D = diameter of pump piston in inches, and H is in feet,

$$H = \frac{\sqrt{DP}}{14.7} \quad (352)$$

As before stated, the favourite valve on a ram pump is the mitre valve of gun-metal with a gun-metal seat. Such valves are, however, only of any

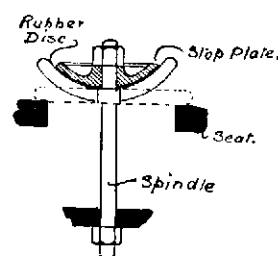


FIG. 573.

efficiency with clean water, for they soon start to leak. For gritty water a gutta-percha top beat and a metal bottom beat is satisfactory. These valves, however, cannot work at a high speed. For piston pumps the disc valve is principally used, having a disc of rubber composition on a gun-metal frame. They are held on the seat by light weights, and are illustrated in fig. 573. It is essential in large pumps not to have one or two large valves, but to substitute an adequate number of smaller ones. It is also important for the engineer to know that when choosing a pump (and that is principally the dealing

with pumps, together with testing, that the reader will be called upon to do), that a pump must always be designed to work well above its rated capacity. Slow-working pumps give more satisfaction, because slip is considerably reduced, wear and tear is reduced, and a good vacuum is easier obtained on steam pumps. Perhaps the term "slip" may want a little explanation. Say we have a ram 10 in. in diameter and 20 in. stroke. $10 \times 10 \times .785 = 78.5$ sq. in. in section area $\times 20$ in. = 1570 cub. in. displacement = 0.908 cub. ft.; viz. in say 1000 strokes the theoretical water pumped will be 908.0 cub. ft. Say, now, we know the actual quantity to be 700 cub. ft. $\therefore \frac{700}{908} = .77 = 77$ per cent. efficiency, or 23 per cent. slip.

A frequent defect in steam pumps is that of pounding. It is often hard to diagnose, but it is most essential that it shall not continue, and the engineer in charge must see to it at once. Its causes are principally—

1. Too much lift of valves.
2. Steam valve not sufficiently advanced.
3. Insufficient clearance.
4. Loose bearings and pins.

Ram pumps are usually packed with hemp gasket soaked in tallow, but many patent packings are now used; they should for domestic water supplies be sheathed with gun-metal. The packing must not be so tight as to score the rods. Piston pumps such as those of the Worthington type, figs. 570-1-2, are packed either with leathers (two cup leathers back to back), cotton, bronze expanding "Ramsbottom" rings, or similar rings of ebonite, kept tight by the water pressure. For pumping sewage by means of other than centrifugal pumps, iron clack valves with leather seats are used. They must work at a slow speed. It is essential to have a strainer at the foot of all suction pipes; it is called the "snorer" or "wind bore." A very long snorer used for sinking purposes is called the sinking slide. What is termed a bucket and plunger pump is one in which a bucket lifts the water on the upstroke, fig. 574, while on the downstroke the plunger will displace it. It therefore becomes double-acting, half the bucket volume being raised at each stroke. The plunger is always at the surface for access to the stuffing box. The suction pipes, which must have foot valves of larger diameter than themselves, must be of ample size, while it is essential that all delivery pipes be of specially good form and of truly cylindrical bore, laid as far as possible with a uniform slope and free from bends. The delivery speed should be limited to 280 ft. per minute, as friction will increase as the velocity squared.

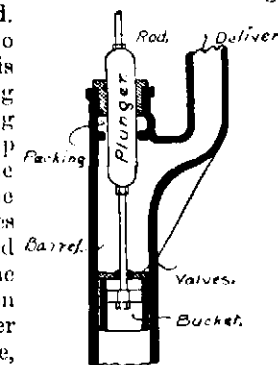


FIG. 574.

We will now turn to the more theoretical side of reciprocating pumps, and provide the reader with an insight to testing the engines under his charge, which is most essential.

Firstly, to ensure makers carrying out their guarantees.

Secondly, to ensure the steam consumption being kept low.

The thermal efficiency of engines and boilers would be somewhat out of place here. Besides, the work wants a little practice. But the necessary things to ascertain are as follows:—

1. The consumption of water in the boiler; that is, measured by the feed if a jet condenser is used, and by the hot well when a surface condenser (the latter affording the best results).
2. The temperature, so as its correct weight may be ascertained; all sources of loss are considered.
3. All auxiliary engines, if possible, to be stopped. If injectors are provided to the boilers, use them in preference to pumps, unless driven off the main engines.
4. All water in boilers should be lowered to the lowest safe working level, and kept the same right through the tests. This is principally to prevent priming, and to ensure dry steam.

5. The pumping engine must be indicated at least three or four times during the tests.

6. The pumps must be kept "solid," and the theoretical amount of work done ascertained. If the engine is lifting pumping rods, as in fig. 568, and these rods are heavier than the water raised, the work done is the same at every stroke, irrespective of the quantity of water, viz. the raising of the rods; in fact, in all pumps, if they are kept solid. The work is the same, no matter what the efficiency of the pump is.

7. For sake of comparison, assume a fair average coal consumption of 10 lbs. of water converted into steam at working pressure per lb. of coal.

8. The duty of a pump is expressed as so many foot-lbs. per cwt. of coal consumed, on the evaporation basis of 10 lbs. of water per lb. of good coal.

The following duties have been obtained in practice:—

		Duty.
Bull engine (colliery) cyl.	100" × 12'	50,265,312
Davey differential engine	36" × 4'	41,360,000
Duplex	22" × 2'	41,360,000
Davey differential	33" × 9'	69,705,390
" compound	52" × 12'	69,705,390
Bull engine	...	35,827,200

and the average results we may expect are—

Pumps working by rods	30,000,000-110,000,000	} per 110 lbs. of coal.
High-class pumping engines	15,000,000-30,000,000	
" " (small)	7,000,000-15,000,000	
Vacuum pumps	3,000,000-10,000,000	

Worthington Pumping Engine Tests.

1. Worthington pump, with compound cylinders, gave an efficiency of 91.5 per cent. average of three trials; that is, proportion of pump horse-power to indicated horse-power.

2. The direct-acting pumping engine at the St Maur station of the Paris Waterworks gave the following results:—

Regular velocity of pump and engine, 354 ft. per minute; water pumped, 2300 gallons per minute, under a pressure of 120 lbs. to 150 lbs. per sq. in. The duty performed was given as 1.984 lbs. to 2.095 lbs. of fuel per H.P. hour in water lifted = 1.764 lbs. per H.P. hour at the fly-wheel shaft, and 1.543 lbs. per hour per I.H.P.

The engine, constructed by Farcot & Sons, is of the Corliss type, and the clearances at each end of the cylinder are less than 1 per cent. The cylinder is jacketed, and the cut-off may be adjusted from 0 to 80 per cent.

Now it is very often the case that pumping engines are gas-driven instead of steam. In this case the indicated horse-power is by no means reliable. They must be tested on the brake, but the diagrams are a useful check. In addition, we have to find very carefully the gas consumption, its temperature and pressure, its calorific value in B.T.U.'s per cub. ft., the jacket temperature, the amount of cooling water used, and other minor details. In fact, we cannot fully describe engine-testing here. The reader must refer to indicator diagrams by C. N. Pickworth, and the testing of engines and boilers by W. W. F. Pullen, for further information; also the standard rules for testing, lately

issued by the Institute of Civil Engineers. The driving of pumps by electric motor and water-power is also now much in vogue; any available water-power should always be taken advantage of, not merely to supply all the power, but to assist the main engines when this cannot be done. Every little helps, and reduces the yearly coal bill. The things which the engineer has to ascertain during the test of a steam pump are as follows:—

1. Number of single strokes of pump per minute.
2. Total head in feet, suction + delivery.
3. Water pumped in gallons per minute.
4. Piston displacement in gallons per minute.
5. Rated displacement " "
6. Actual capacity, gallons in 24 hours.
7. Steam-pressure gauge.
8. I.H.P. steam end.
9. I.H.P. water-end.
10. Developed horse-power = D.H.P.
11. Dry steam per hour (pounds).
12. " I.H.P. hour (").
13. B.T.U.'s per D.H.P. hour (No. 10) (above ideal feed temperature).
14. Mechanical efficiency = $\frac{\text{I.H.P. water cylinder}}{\text{I.H.P. steam cylinder}}$
15. " " = $\frac{\text{I.H.P. steam cylinder}}{\text{D.H.P.}}$
16. Slip per cent. = $\frac{\text{line 4} - \text{line 3}}{\text{line 4}} \times 100$
17. " " = $\frac{\text{line 5} - \text{line 3}}{\text{line 5}} \times 100$

18. Duty in million ft. lbs. per cwt. of coal burnt, or per 1,000,000 B.T.U.'s.

As we are laying more stress on the actual testing of pumps than mere engines, a few figures may be of interest to the reader, and guide him in his future practice. The test is of a small 7½ in. × 12 in. simplex pump with steam-actuated valve gear, and the figs. obtained during the actual test were as follows:—

1	5	9.2	13.7	20	28	41	48.6	61.2	72.2
2	204	196	214	208	219	219	220	230	240
3	10	18	25	38	54	78	93	117	139
4	10.05	18.05	26	40	56.5	81	96.5	122	145
5	11.72	20.45	29.7	44.9	62.75	86.75	103.6	130.5	154.2
6	14,400	25,920	36,000	54,720	77,760	112,300	133,920	168,480	200,160
7	34	39	43	45	46	51	54	61	60
8	55	1	1.5	2.25	3.2	4.8	5.8	7.38	8.86
9	51	.95	1.4	2.1	2.95	4.35	5.35	6.85	8.26
10	5	.9	1.35	2.0	2.85	4.20	5.17	6.63	8.0
11	235	293	364	461	511	655	721	860	889
12	424	293	213	205	159.5	136.3	124.3	116.5	109
13	7975	5550	4440	3965	3075	2680	2400	2230	1899
14	921	.95	.933	.933	.923	.925	.922	.917	.932
15	.907	.90	.90	.91	.89	.875	.89	.897	.902
16	.497	.278	3.85	5.00	1.42	3.7	3.63	4.1	4.13
17	14.85	11.97	15.81	15.35	13.95	10.10	10.25	10.35	9.86
18	4.14	5.95	7.15	8.35	10.71	12.34	13.75	14.8	17.35

From the above figures the curve illustrated in fig. 575 has been plotted. Such curves are very useful on such tests, and should always be plotted.

We now pass on to an entirely different type of pumping engine known as the centrifugal pump; there are no plungers or pistons, and the pump does not work by displacement. It is simple in the extreme, especially the original type before the latest improvements were made. There is no doubt that for lifting large volumes of water to moderate heads that the centrifugal pump stands supreme, especially for liquids such as sewage and dirty, gritty water; they have no valves to get out of order. The essential parts are the

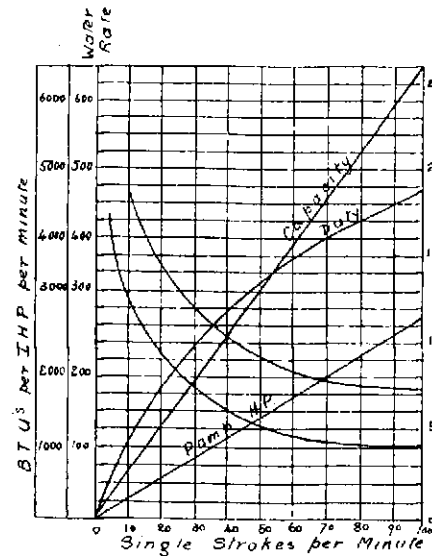


FIG. 575.

purpose. A good form of modern single-stage pump is shown in fig. 576, designed to lift 450 cub. ft. per minute to a height of 250 ft. The spiral volute chamber is for the purpose of converting kinetic energy into pressure energy. The pump is designed for a motor drive, and the following are the calculations in getting a rough idea of the principal dimensions.

Design for a Single-stage Centrifugal Pump.

Capacity 450 cub. ft. per minute = 2812.5 gallons. Head 250 ft. (See City and Guilds Exam. in Mechanical Engineering Honours Grade, 1909.)

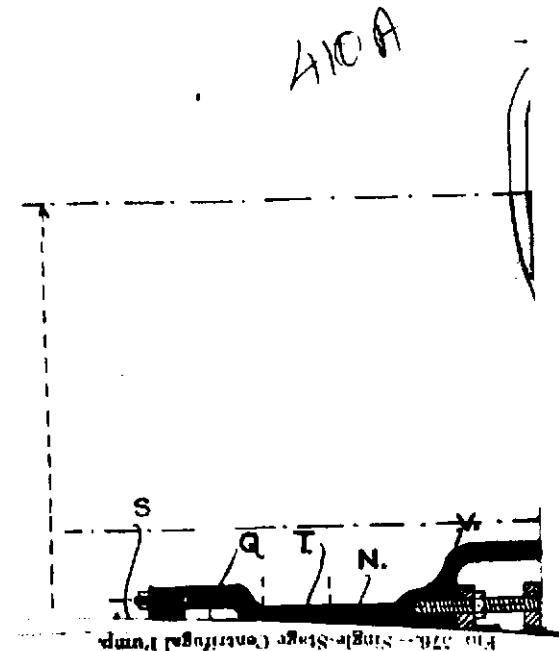
Under these circumstances we shall fix the speed at 950 revs. per minute, because the modern tendency is to drive direct by single or polyphase or continuous-current motors.

First of all, to get the correct circumference of the fan, and hence the diameter, we use the equation

$$c = \frac{\{(8 \sqrt{H}) + K\} \times 60}{\text{RPM}} \quad (353)$$

casing, usually of cast iron, and the impeller, sometimes of cast iron and sometimes of bronze. In this impeller are curved vanes usually 6-8 in number. The water entering round the shaft at the centre or "eye" of the impeller is at once acted upon by the centrifugal force set up by the impeller and flung outwards, so to speak, causing pressure energy at the vane tips and a vacuum at the centre. The curves of the vanes were in the original pumps spiral. Modern makers are now designing them to be nearly tangential at the inlet to radial at the periphery.

Unlike displacement and lift pumps, centrifugal pumps must work at their rated capacity (neither more nor less), because if this is departed from, the vane angle, which is the important part of the design, becomes quite unsuitable to the



H = head in feet (+ 20 per cent. for friction) = 250 + 50 = 300
and K = a coefficient value in this case = 10

$$= \frac{\{(8 \sqrt{300}) + 10\} + 60}{950} = \frac{\{8 \times 17.32\} \times 10}{950} \times 60 = 9.37$$

$$= \frac{9.37}{\pi} = \text{say } 3 - 0 \text{ in. as diameter of impeller.}$$

The modern tendency as regards the blades appears to be favouring more than the older pumps, which usually had 6. Say we supply 10 blades. The suction and discharge pipes may be made of the same diameter, water being considered as incompressible. Formula used

$$= D = 7.57 \sqrt{V} \quad . \quad . \quad . \quad . \quad . \quad (254)$$

which when V = volume of water in cubic feet which is pumped per minute = $7.57 \times \sqrt{450} = 7.57 \times 21.21 = 16$ say, delivery and discharge pipes are made 16 in. in diameter.

The next thing to do is to settle the width of the mouth of the fan. Use the equation

$$W = \frac{A}{c_1} \quad . \quad . \quad . \quad . \quad . \quad (355)$$

A being the area of the discharge pipe in square feet and c_1 = circumference of the wheel, also in feet, less the 10 blades $\frac{3}{4}$ in. thick = 7.5 in. $\therefore$ area of discharge pipe

$$= \frac{201.6}{144} = 1.395 \text{ sq. ft.}$$

and

$$c_1 = 9.37 - .6 = 8.77 \text{ ft.}$$

$$\therefore \frac{1.395}{8.77} = .159 \text{ ft.} = 1.78 \text{ in.}$$

$\therefore$ width of fan at mouth = 1.78 in.

The sectional area of the guide throats should be equal to that of the suction pipe, viz. 201.6 in. By trial and error we find diameters of 13 in. and $6\frac{1}{2}$ in. satisfy our requirements, because we know that the area of an annulus = $(D - d)(D + d) \times .785 = 19.5 \times 6.5 \times .785 = 99.5$ sq. in. on each side, because the pump has two eyes. $\therefore 99.5 \times 2 = 199$ sq. in., which is a very close result to 201 sq. in.

The horse-power (approximately) required to drive the pump will be

$$\frac{G \times 10 \times H \times \frac{3}{2}}{33000} \quad . \quad . \quad . \quad . \quad . \quad (356)$$

G being delivery in gallons per minute and H = head in feet

$$= \frac{2812.5 \times 10 \times 250 \times \frac{3}{2}}{33000} = 319.625.$$

Horse-power to be provided = 320.

- For the shaft diameter we have

$$D = \sqrt[3]{\frac{83 \text{ H.P.}}{\text{R.P.M.}}} \quad (357)$$

$= \sqrt[3]{\frac{83 \times 320}{950}} = \sqrt[3]{28.2} = \text{say a } 3\frac{1}{4} \text{ in. solid shaft.}$ But the pump will run at a very high speed, and a large diameter shaft will give more efficient lubrication, and is less liable to run hot. Therefore we are going to substitute a hollow shaft, having internal diameter = 3 in. and the external diameter = 6 in. It will be of nearly double the strength required by working out the equation

$$R = \frac{\sqrt[3]{r_1^4 - r_2^4}}{r_1} \quad (358)$$

It being radius of solid shaft and r_1 and r_2 the external and internal radii of the hollow one

$$= 1.5 = \sqrt[3]{\frac{3^4 - 1.5^4}{3}} = \frac{\sqrt[3]{81 - 5.025}}{3} = \sqrt[3]{25.33}$$

$= 2.92$, which is > 1.5 .

The length of the bearings should be three times the outer diameter of the shaft, viz. $6 \text{ in.} \times 3 \text{ in.} = 18 \text{ in. long.}$

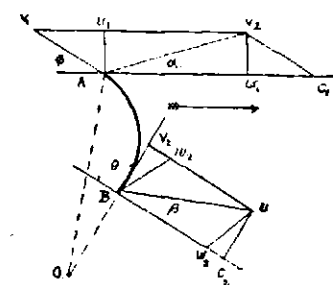


FIG. 577.

The description of the pump is as follows (fig. 576):—

A is the bearing.

BB bronze lubricating rings.

C the oil reservoir with draw-off cock.

DD packing glands.

EE = gun-metal bushes turned from the solid, forced in by hydraulic pressure and finished off in the lathe.

F is the impeller, cast from phosphor bronze of a special hard quality, having an ultimate tensile strength of 20 tons per sq. in. and an elongation of 15% in 10 in. It

should receive a high polish, because experiments prove a distinct economy by the use of such a polished impeller. It is fixed in place by a nut X on a plus thread Y.

The pump is self-balancing owing to the double inlets. HH are small valves on copper pipes GG. By adjustment of these valves any leakage past the bushes EE is drawn into the delivery pipe, or the reverse process may take place, that is, water leak into the pump, and prevent air from being drawn in. JJ is the pump cover bolted on. Faced joints or rubber packing may be used.

KK is the spiral volute chamber and diffuser. It turns the kinetic into pressure energy. L is a flange coupling; 6 bolts are provided. It is faced up in the usual manner. M is the hollow shaft. N is a bush in which the end of the shaft is reduced in diameter works. It is 4 in. in diameter, and bored hollow to 3 in. On the shaft and bush are turned rings PP. They are to prevent any undue side-play. Q is a special arrangement so that the oil in the chamber R is continually forced into the small recess T at the end of the shaft. It finds its way by means of small holes into the rings PP, and so provides a continuous lubrication. S is an inspection cover.

VV Are two removable casings.

Regarding the general theory of the centrifugal pump, refer to the diagram fig. 577.

r_1 = radius at A and c_1 = velocity here.

r_2 = radius at B and c_2 = " "

$Bc_2 = c_2$ = velocity at entry.

$Ac_1 = c_1$ = velocity at discharge

$Bw_2 = w_2$ = velocity of whirl at entry } relative.

$Aw_1 = w_1$ = " " discharge.

Bu_2 and Au_1 = radial components at entry and discharge.

Then the work done per lb. of water during its passage through the disc

$$= \frac{c_1 w_1}{g} \quad (359)$$

and the loss of head

$$= \frac{u_1^2}{2g} \quad (360)$$

when there is no diffuser, but if there is a discharge pipe of increasing diameter it is equal to

$$\frac{D^2}{2g} \quad (361)$$

D being velocity of discharge in ft. per second.

Now the larger the ratio of the distance from centre to point of entry to the distance from centre to point of entry into the volute, the greater will be the efficiency of the pump, but the heavier the pump will be.

This is an important consideration in design.

Now if Z = the number of impeller arms,

b = breadth of the impeller in feet,

a = distance between the vanes in feet,

Q = discharge in cubic ft. per second,

we have u_2 = radial component at entry

$$= \frac{Q}{Z(ab)} \quad (362)$$

and the loss of head at entry

$$= \frac{1}{2g} \times (c_2 - u_2 \cot \theta)^2 \quad (363)$$

But when

$$c_2 = u_2 \cot \theta \quad (364)$$

there will be no such loss of head.

Now the usual practice is to use a discharge pipe of increasing diameter. When, however, it is not used, there will be the least loss of head when

$$v_4 = \frac{1}{2} w_1 \quad (365)$$

The hydraulic efficiency is calculated

$$h = \frac{gH}{c_1 w_1} \quad (366)$$

Now, according as the angle ϕ is increased, the value of c_1 in relation to

$\sqrt{2gh}$ will decrease, and so will the hydraulic efficiency, for w_1 decreases as ϕ decreases.

Again, there will always be a certain loss of head by shock at entry

$$= I_s = c_2 - u_2 \cot \theta \quad (367)$$

Regarding the effect the type of vane has, we can have either radial vanes or vanes with a backward or forward curve. If the vane is radial,

$$c_1 = w_1 \quad (368)$$

If inclined backwards (Appold vane),

$$c_1 > w_1 \quad (369)$$

If inclined forwards,

$$c_1 < w_1 \quad (370)$$

With backward vanes we have a rapid change of direction in the fluid. They are easily adapted to the involute form. They cause " α " to be large (*vide* diagram 577).

The straight vane is, of course, the simplest to manufacture, but is not much used, because the inner blade angle determines the size of the outer one.

Forward vanes allow of slow speeds, but the backward vanes, which require a high speed, are now usually used. They cause α to be small in value (*vide* diagram 577).

Now if we take the angle $\phi = 90^\circ$, $c_1 = w_1$, because $\cot 90^\circ = \text{zero}$. $\therefore$ in this case instead of h (hydraulic efficiency) being equal to

$$\frac{gH}{c_1 w_1}, \text{ it is equal to } \frac{gH}{c_1^2} \quad (371)$$

and generally the larger value ϕ has, the slower will be the pump speed. Now if V = the absolute velocity is known, we can find the theoretical gauge pressure at discharge in feet-head

$$P = \frac{r_2^2}{2g} \left(1 - \frac{r_1^2}{r_2^2} \right) \quad (372)$$

r_2 being the distance from centre to point of entry into volute. r_1 has been given. The actual gauge pressure (and a gauge should be provided on the discharge pipe quite near the pump casing) will not be much more than half this value, showing a diffuser efficiency of about 55 per cent. The value of the diffuser is principally to allow of a varying discharge at a constant head. Again in some calculations to follow we shall require the manometric efficiency. Its value is

$$\mu = \frac{gH}{c_1^2} \quad (373)$$

(*vide* Equation 371)

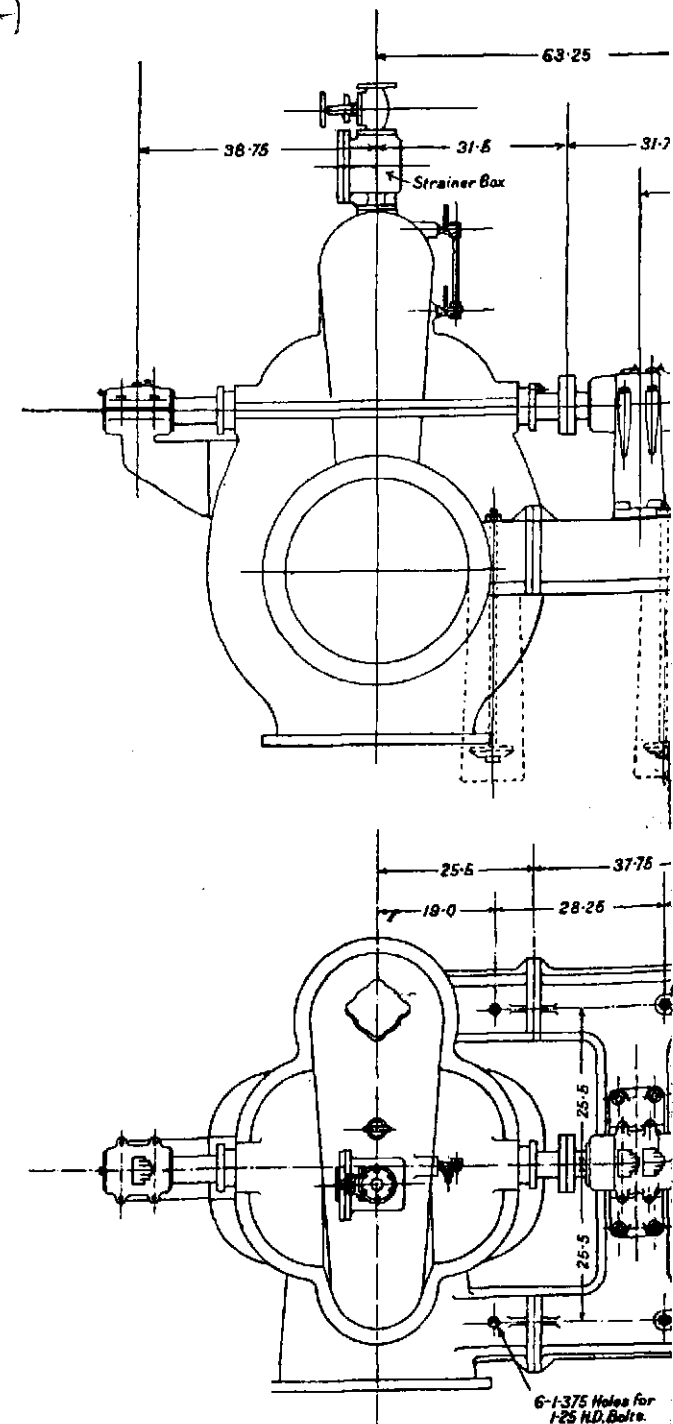
while the volumetric efficiency

$$= 8 \frac{Q}{c_1 r_1^2} \quad (374)$$

For pumps which are required to discharge variably or less than the normal $\phi = 90^\circ$ is quite unsuitable, because at slow speeds the pump would probably not work at all; when, however, ϕ is as low as 15° there will be

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$\sqrt{2gh}$ will decrease, and so will the hydraulic efficiency, for w_1 decreases as ϕ decreases.

Again, there will always be a certain loss of head by shock at entry

$$= I_s = c_2 \cdot u_2 \cot \theta \quad (367)$$

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and generally the larger value ϕ has, the slower will be the pump speed. Now if $V =$ the absolute velocity is known, we can find the theoretical gauge pressure at discharge in feet-head

$$P = \frac{v^2}{2g} \left(1 - \frac{r_1^2}{r_3^2} \right) \quad (372)$$

r_3 being the distance from centre to point of entry into volute. r_1 has been given. The actual gauge pressure (and a gauge should be provided on the discharge pipe quite near the pump casing) will not be much more than half this value, showing a diffuser efficiency of about 55 per cent. The value of the diffuser is principally to allow of a varying discharge at a constant head. Again in some calculations to follow we shall require the manometric efficiency. Its value is

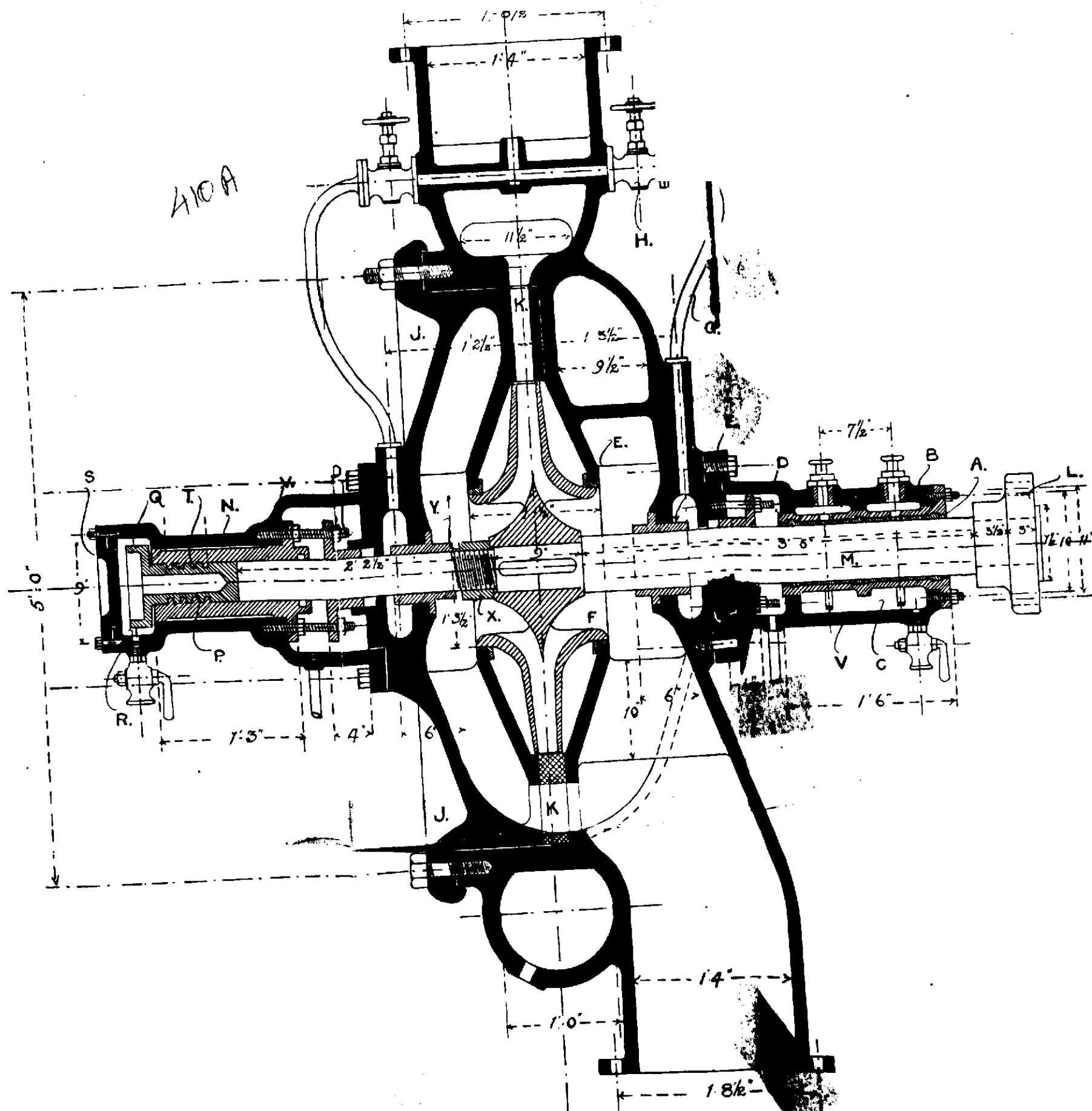
$$\mu = \frac{gH}{c_1^2} \quad (373)$$

(*vide* Equation 371)

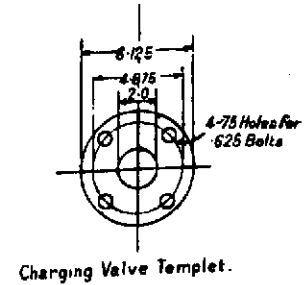
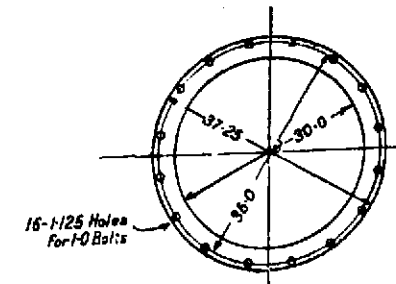
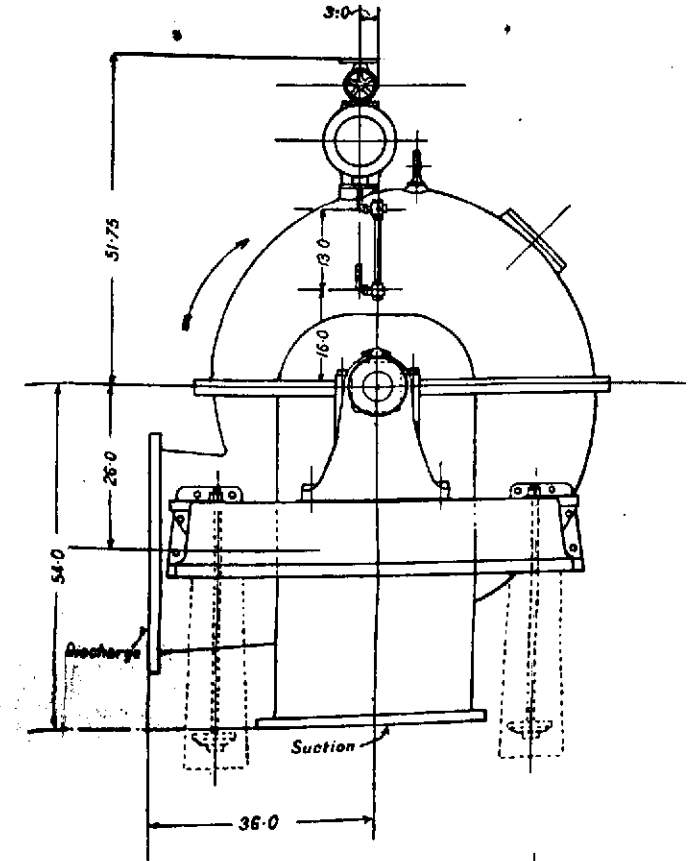
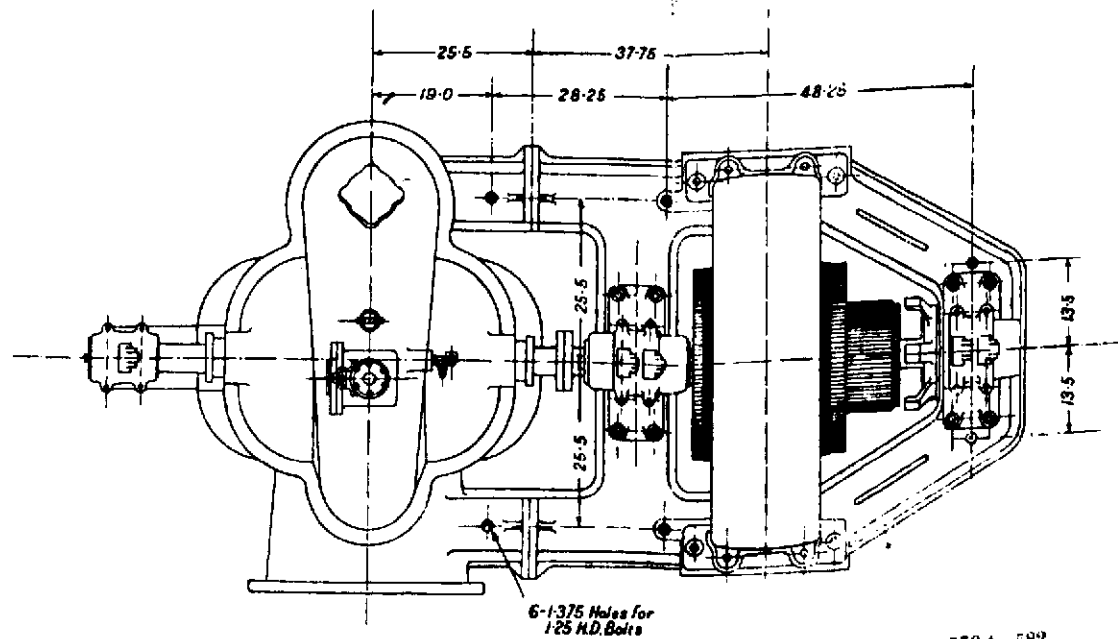
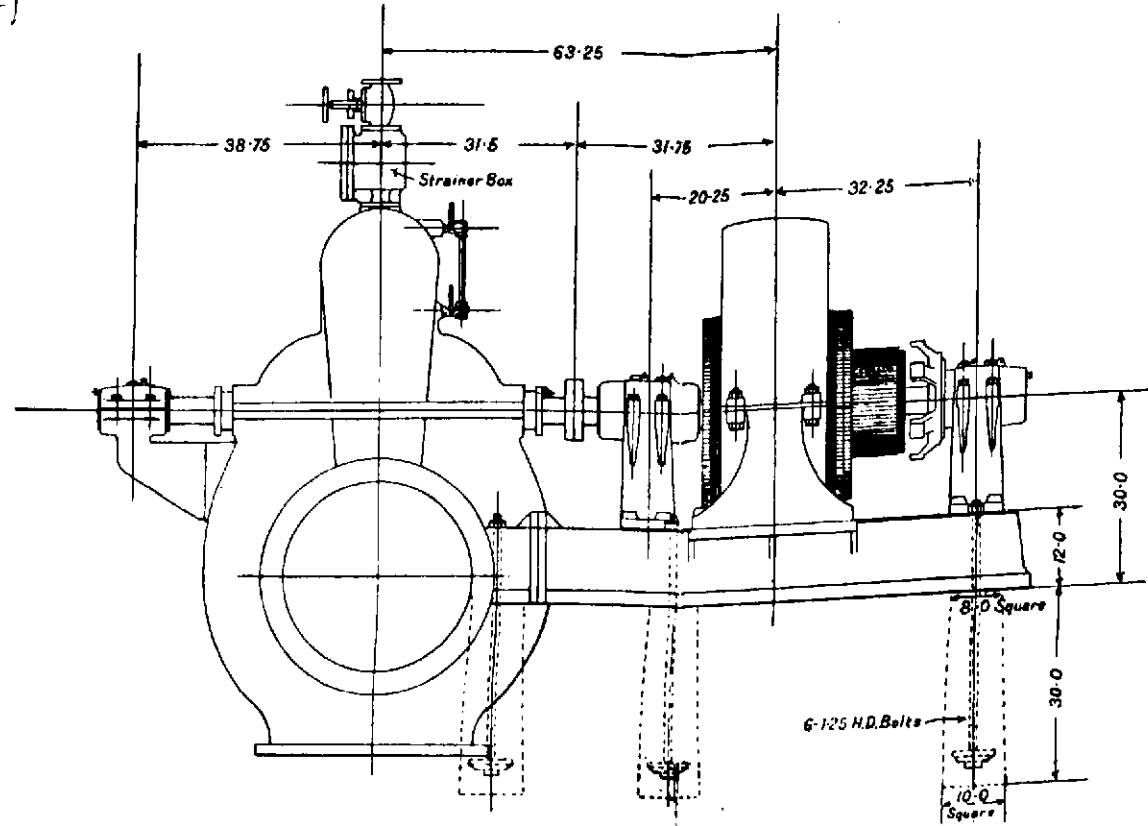
while the volumetric efficiency

$$= 8 \frac{Q}{c_1 r_1^2} \quad (374)$$

For pumps which are required to discharge variably or less than the normal $\phi = 90^\circ$ is quite unsuitable, because at slow speeds the pump would probably not work at all; when, however, ϕ is as low as 15° there will be



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FIGS. 578 to 582.

considerable fan friction, but less, however, in the volute and discharge pipe. The balancing of the pumps is essential. It is carried out in various ways according to the design, but usually it resolves itself into equalising the pressures on each side of the fan, and the first and last impellers of multi-stage pumps. The bearings are also of special design, a certain amount of end thrust being provided for.

With electrically-driven pumps polyphase motors are now often used. Electrical engineers seem to favour 50 cycles as the periodicity, which with 3-phase motors causes 1450 R.P.M. to be a favourite speed.

To find the speed of any similar motors we have the formula

$$\text{RPM} = \frac{60 \times \text{number of pairs of poles}}{\text{number of pairs of poles}} \quad (375)$$

less .05 for slip. As a good example of a modern centrifugal pump electrically driven I am able, by the courtesy of Messrs W. H. Allen & Sons, Bedford, to illustrate a large machine which was designed for the Middle Dock, South Shields. It is a 30-in. pump driven by a 160 B.H.P. continuous-current motor, running about 270 R.P.M. On the top of the pump casing the reader will observe a water-glass gauge. This accessory should be on all pumps, especially sewage pumps, as it enables the engineer to see that the pump is quite full of water, and not "sucking wind." For keeping the pump full of water (which is absolutely necessary in centrifugal pumps, because they will not start up unless fully charged) a retaining-valve is most essential on the suction pipe, and at the same time the suction should be as short as possible; 7-8 ft. is ample; but, after long standing, the pump is very liable to get dry, and a charging funnel must always be provided on the top of the casing also. It is clearly shown in the pump illustrated in figs. 578-9, 580-1-2. The pump is typical of modern practice.

As an example of a steam-driven centrifugal pump of the type used for raising sewage and other large volumes of water, I have given the set of pumping machinery designed by Messrs W. H. Allen & Sons, Bedford, for the Harima Dock.

There is a compound engine, having cylinders 14 in. and 22 in. in diameter $\times$ 13-in. stroke, running at 220 revs. per minute under a steam pressure of 100 lbs. per sq. inch. Two 24-in. pumps are direct-coupled to the engine, one on each side, while the discs of the pumps are 42 in. in diameter. On the whole, it may be said to reflect credit on the designers, and to be very compact and self-contained. It also illustrates modern practice well. The ejectors for charging the pumps at starting are clearly shown. The engine and pumps are illustrated in figs. 583, 584, and 585.

We now have another type of centrifugal pump to consider, the multi-stage pump, so much in vogue now. There is no doubt that these pumps have given highly satisfactory results for large volumes and high lifts. There are many makes on the market, of which the larger percentage seem to be continental. A section of such a pump is illustrated in fig. 586. It does not require much explanation after what we have said in reference to the single-stage pump. The wheels are usually of bronze, and the pumps motor-driven or driven by high speed gas, steam, or oil engines of the vertical type. The general arrangements of such a pump, motor-driven, are illustrated in fig. 587, showing a Worthington pump with 4 stages. I am indebted to the company for the drawings. Before leaving the subject of centrifugal pumps, which deserve the attention of all engineers, we might

The ejectors are usually placed in brick chambers, or in bad ground in cast-iron tubbing. A general arrangement of ejectors is shown in illustrations 589, 590, and 591.

The writer has seen similar machines at work, and considers that

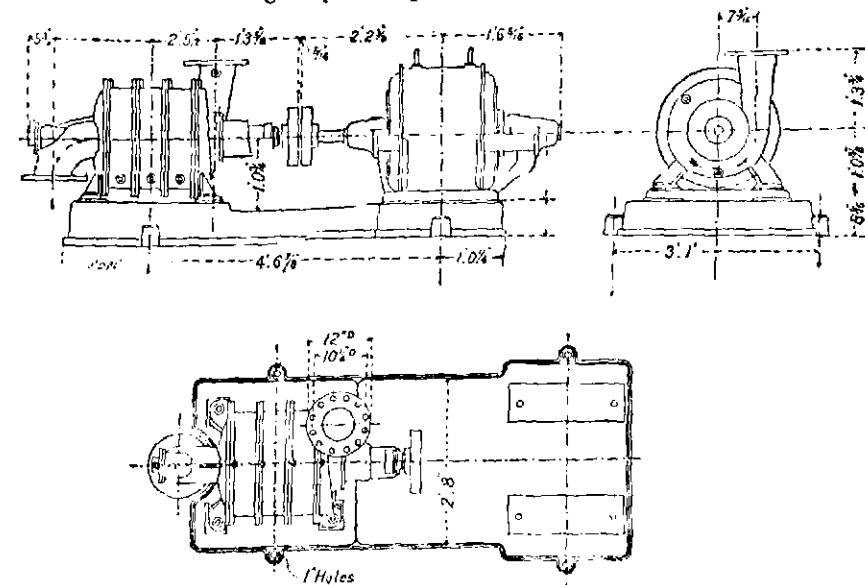


FIG. 587. — Four-stage Turbine Pump and Motor on Bedplate.

stroke of piston. 15,000 gals. per hour = $\frac{15000}{60}$ = 250 gals. per minute against

As the efficiency of a geared pumping engine is only about 66 per cent., we must add a third, or 24 H.P., to the theoretical power, making a total of 96, which is the indicated horse-power that the engine must give out. Steam pressure, 45 lbs.—say 40 lbs. on piston, or 55 lbs. absolute. Cut-off, say $\frac{3}{4}$ stroke. Constant for expanding steam at a cut-off of $\frac{3}{4}$ stroke = 0.968. $55 \times 0.968 = 53.24$ lbs. Back pressure = $15 + 3 = 18$ lbs. $53.24 - 18 = 35.24$ lbs. Diagram factor of 0.9 to allow for losses. $35.24 \times 0.9 = 31.716$ lbs., say

32 lbs. M.E.P. Now $\frac{96 \times 33000}{420 \times 32} = 236$ sq. in., which is the area of cylinder required. This may be either one or two cylinders, and works out at about 18×18 in. single-cylinder, or about 13×18 in. double-cylinder engine, running at 140 revs. per minute in either case.

Or, take the case of four $6\frac{1}{2}$ -in. rams with a 2-ft. stroke at 25 revs. per minute.

Case (2). Same duty as before, but engine-driven, 2 ft. stroke and running at 25 revs. per minute. Steam pressure, 45 lbs. per sq. in. In direct engine-driven pumps the mechanical efficiency should be about 90 per cent. Water horse-power as before = 72. Add 10 per cent. = 7.2, say 8. Then engine must furnish 80 I.H.P. when running at 25 revs. per minute, or a piston speed of $25 \times 2 \times 2 = 100$ ft. per minute. M.E.P. = 32 lbs. Proceeding on similar

lines to Case (1) we get $\frac{80 \times 33000}{32 \times 100} = 825$ sq. in. cylinder area, which gives either one 33×24 in. stroke single-cylinder engine, or one 23×24 in. stroke double-cylinder engine, running at 25 revs. per minute, and cutting off at $\frac{2}{3}$ stroke in either case. These cylinders come out rather large in diameter, but this is due to the low steam-pressure in both cases, and is further aggravated by the low piston speed in Case (2). In all cases the area lost by piston rods has been allowed for, and the diameters given in "round figures."

Take now the case of driving a centrifugal pump by gas engine, under the following conditions:—

All pipes $3\frac{1}{2}$ in., steam pipe 13-ft. suction, 1 elbow, then 90 ft. of pipe rising 1 in 25, then two more elbows before the pump is reached; speed of pump 1260 R.P.M.; duty 4000 gals. per hour; two elbows on delivery.

$$\text{I.H.P.} = \frac{P \times H_{\text{mano.}}}{3600 \times 550 \times \eta_t},$$

where I.H.P. = indicated horse-power required (measured in gas-engine cylinder); P = lbs. of water delivered per hour; $H_{\text{mano.}}$ = manometric head (i.e. suction head + pressure head + friction head + loss of head through delivery at a certain velocity); $3600 = 60 \times 60$ = number of seconds per hour; 550 = number of ft.-lbs. per second = 1 H.P.; η_t = total mechanical efficiency of apparatus under consideration. P in the present case is 4000 gals. per hour, or 40,000 lbs. per hour.

$H_{\text{mano.}}$ = Suction head	.	.	.	.	= 16.6
Pressure head	.	.	.	.	= 18.0 (given)
Friction head (h_f)	.	.	.	.	= 2.94
Loss at outflow	.	.	.	.	= 0.11
					37.65 ft.

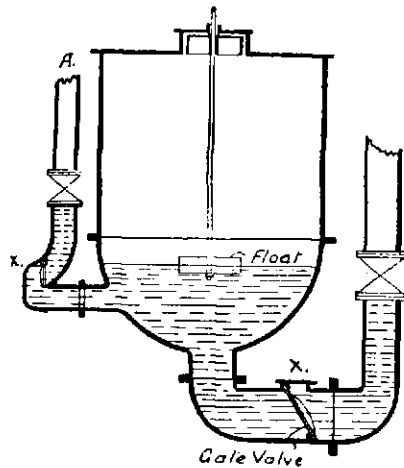
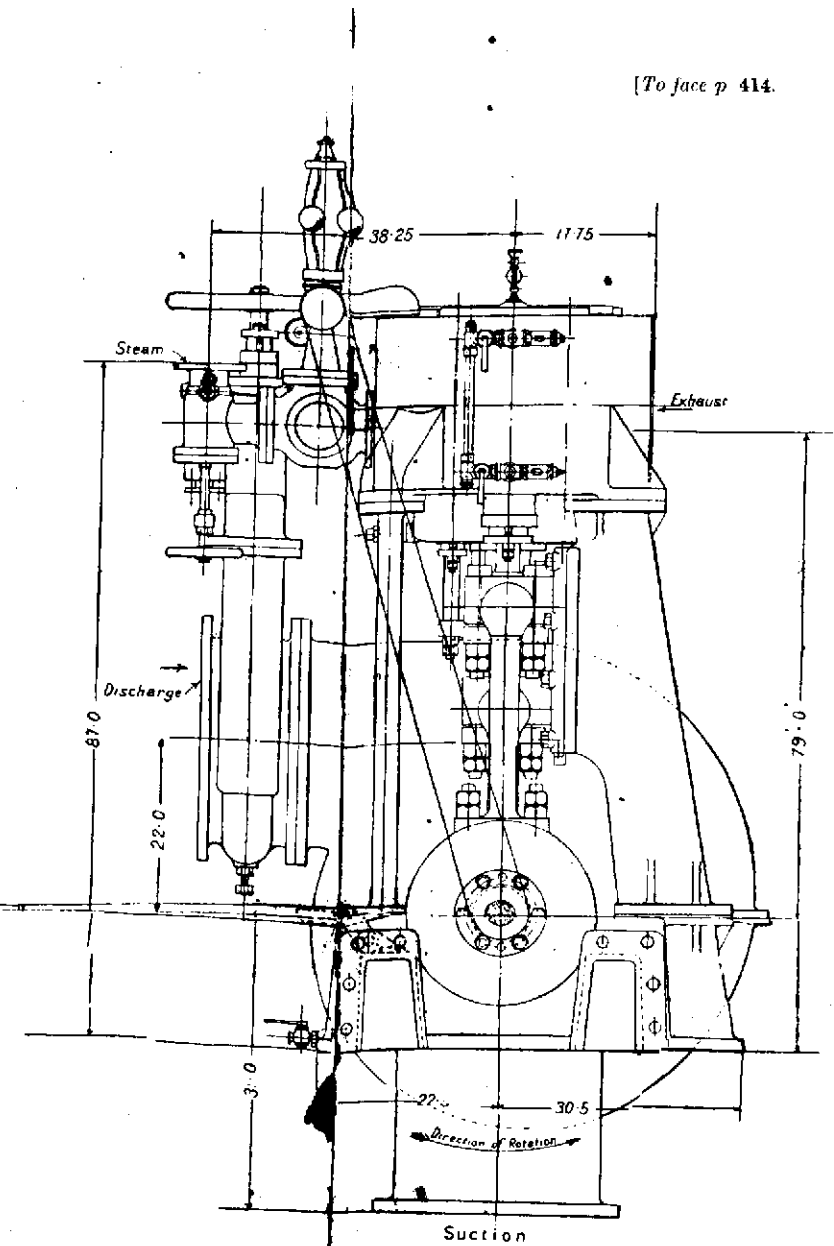
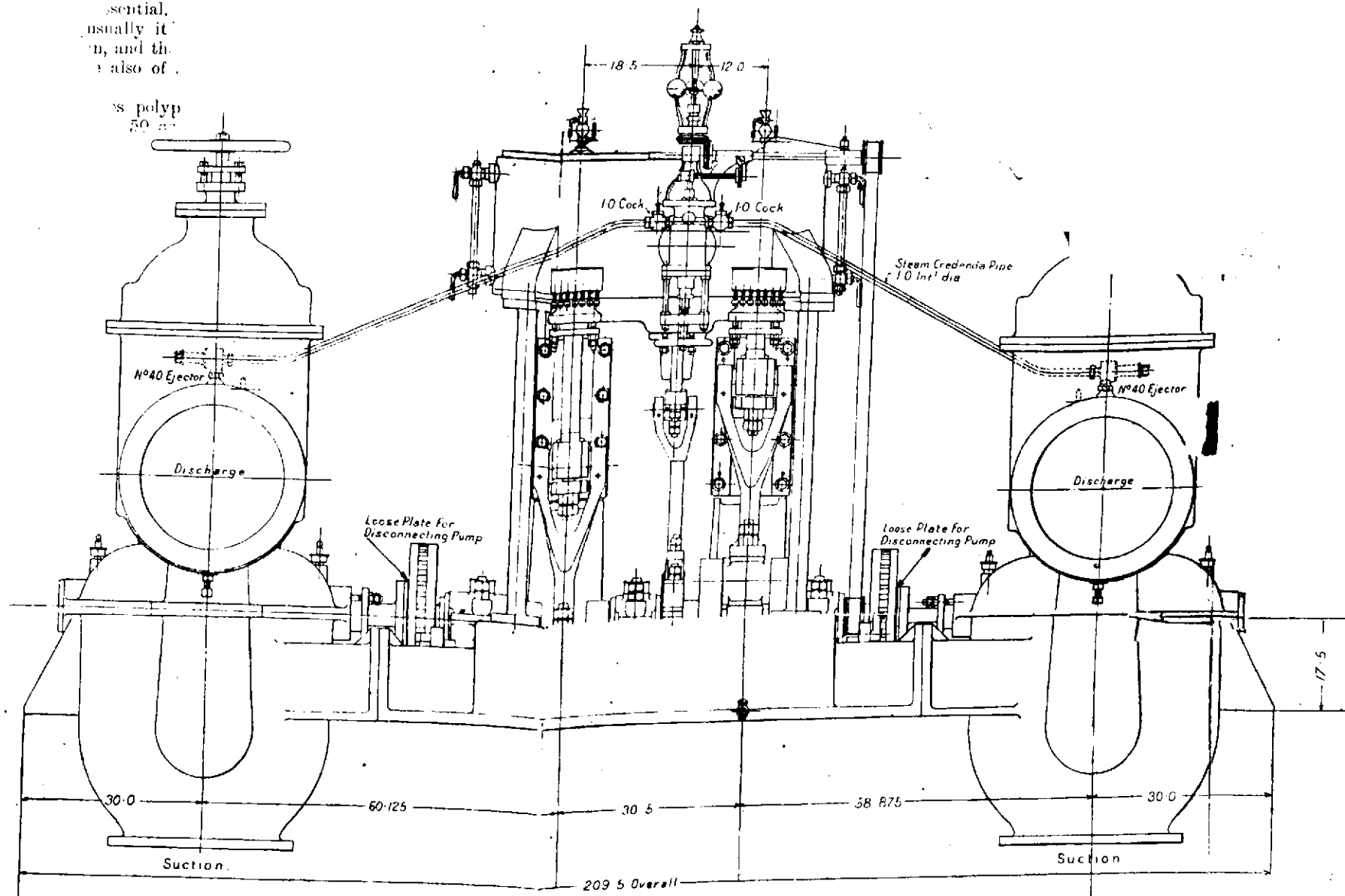


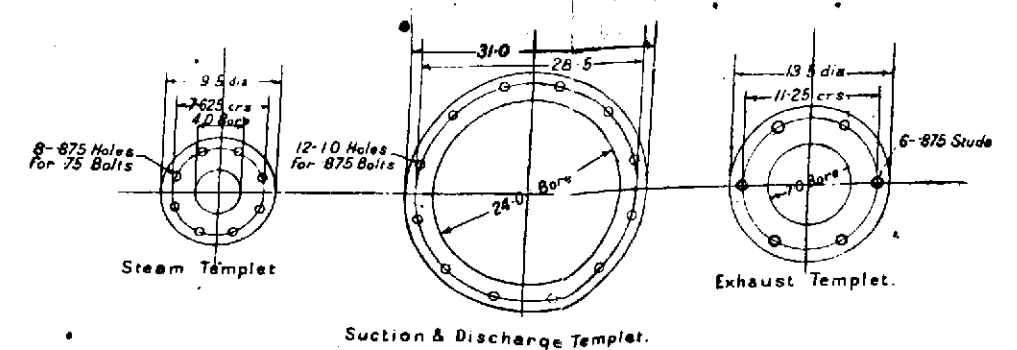
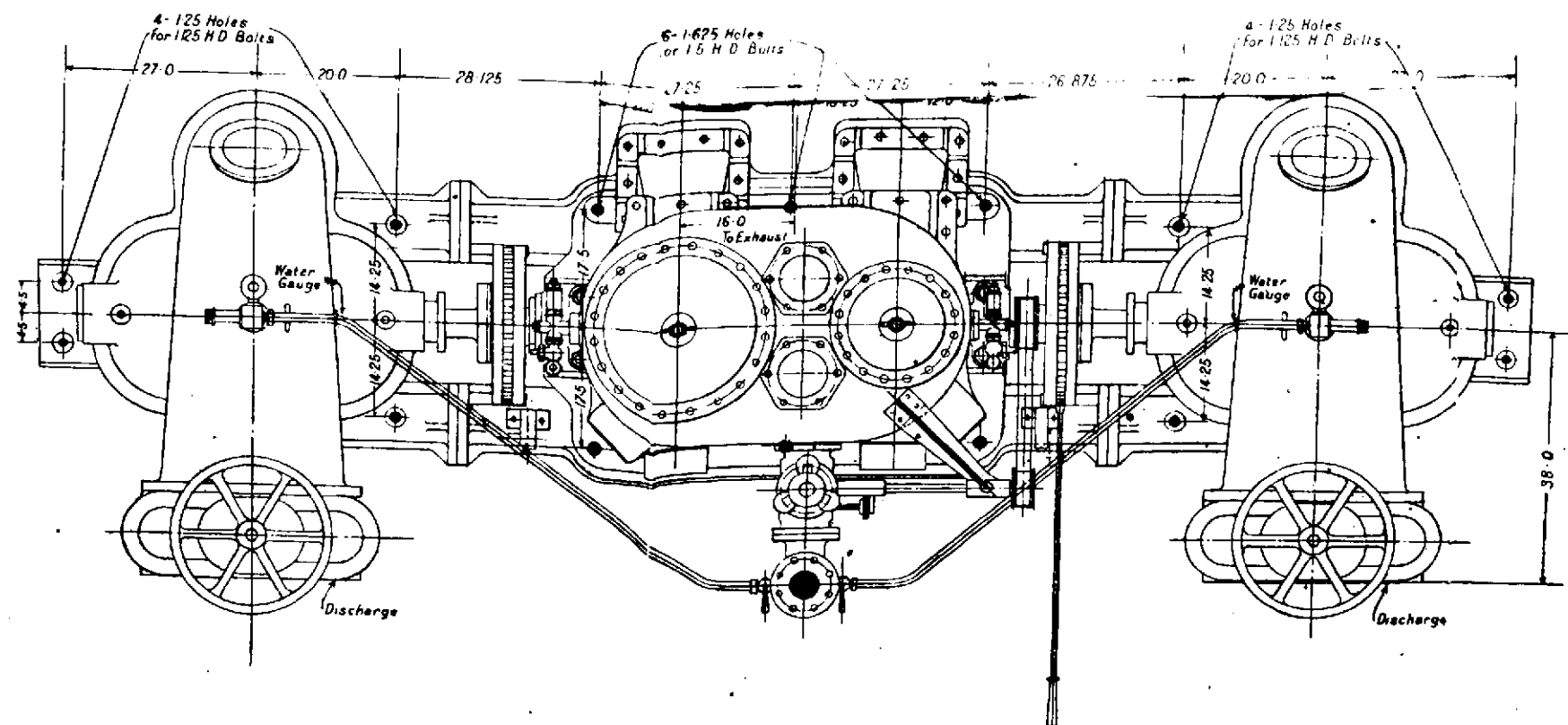
FIG. 588.

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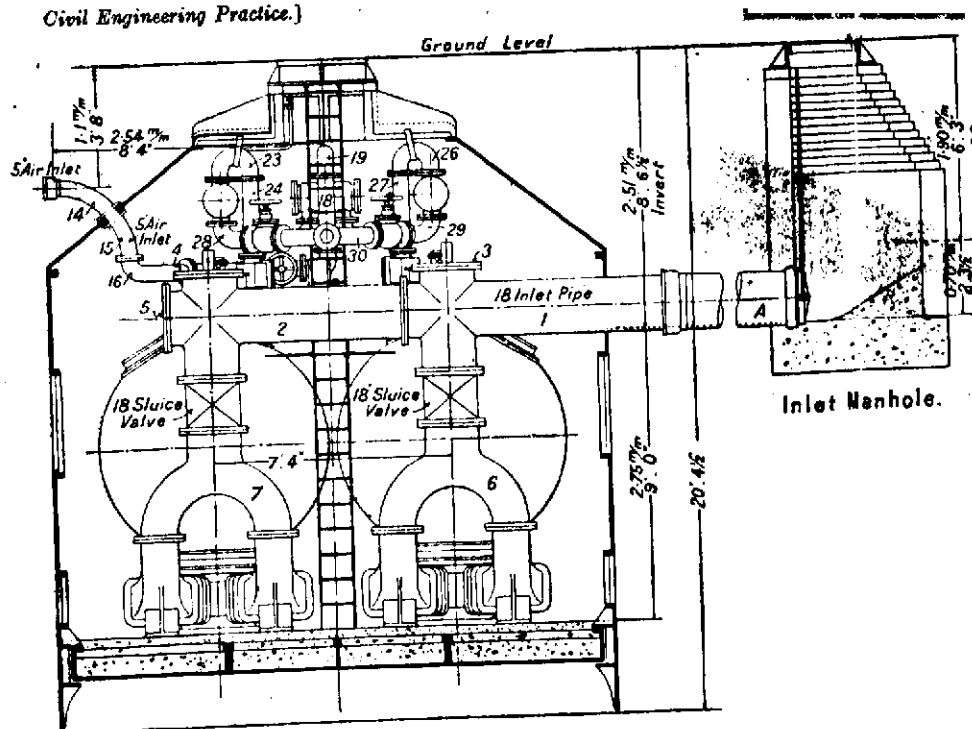
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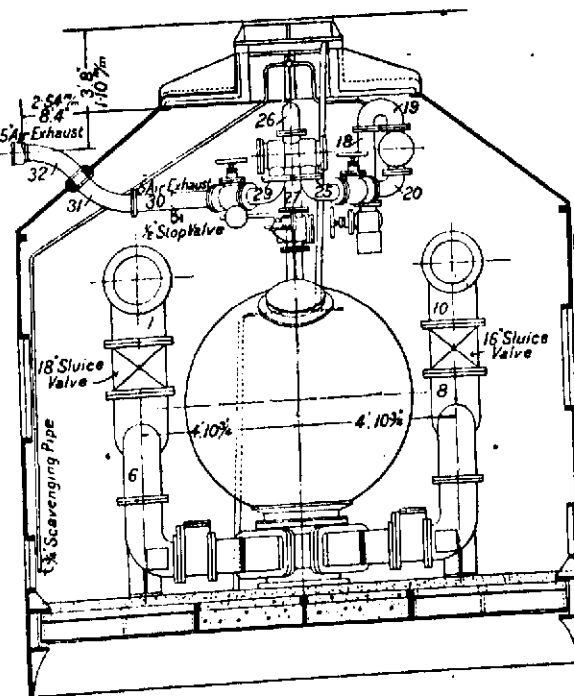
End Elevation, with one Pump Removed



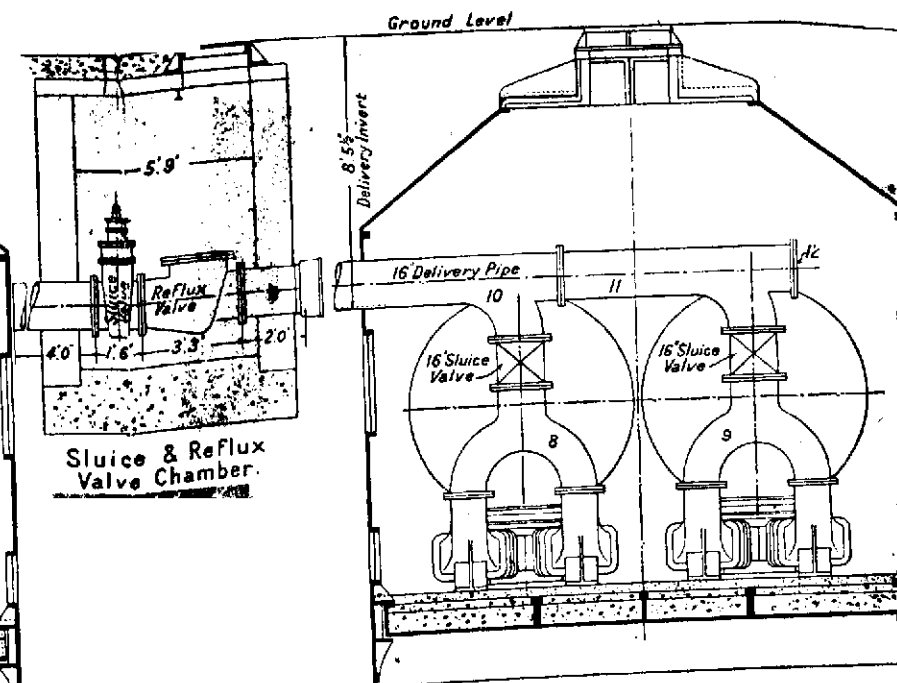
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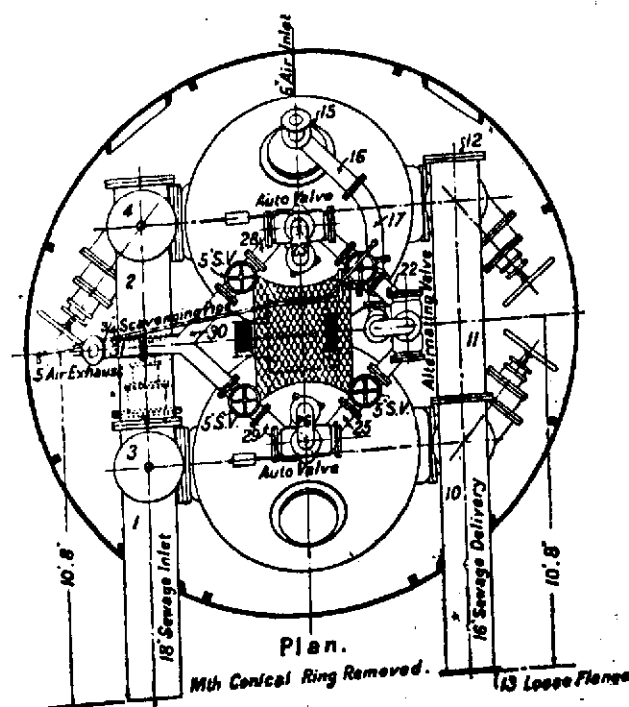
Elevation-Inlet Side.



Elevation.



Elevation-Delivery Side.

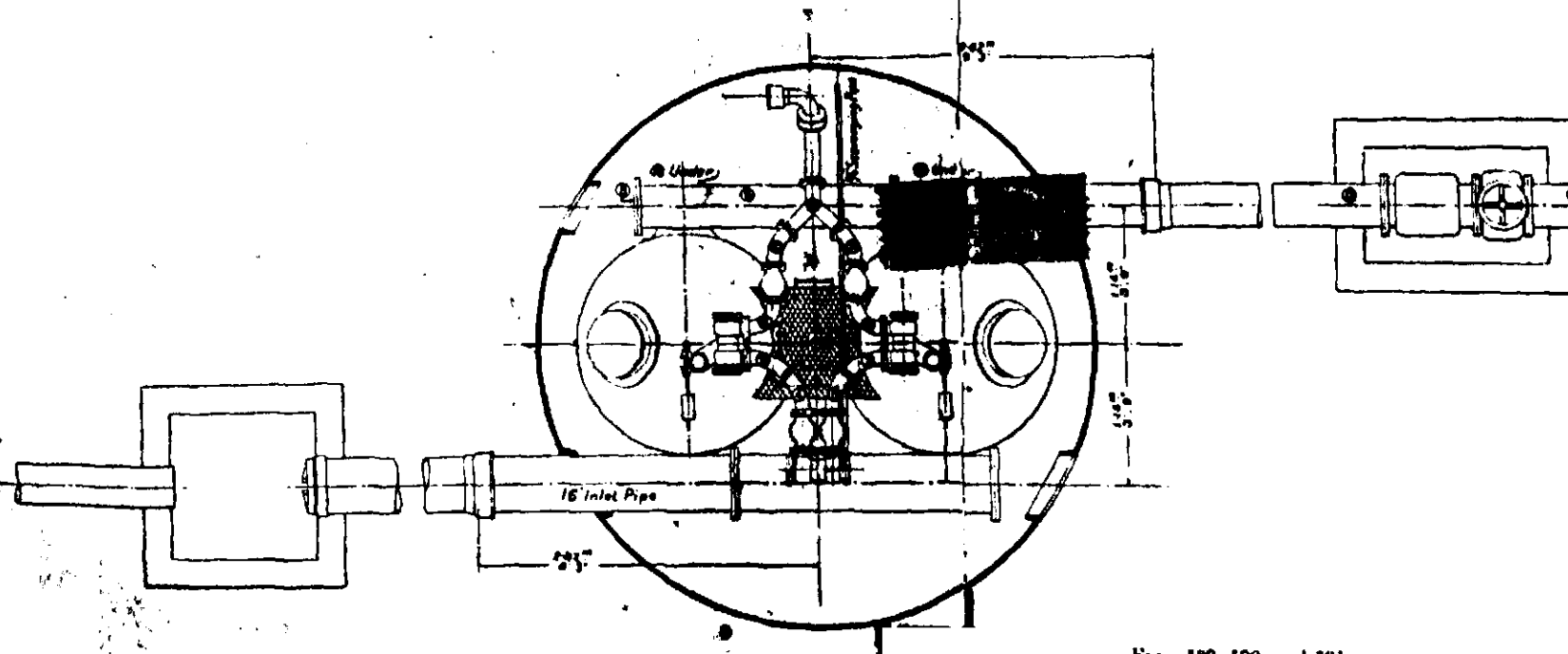
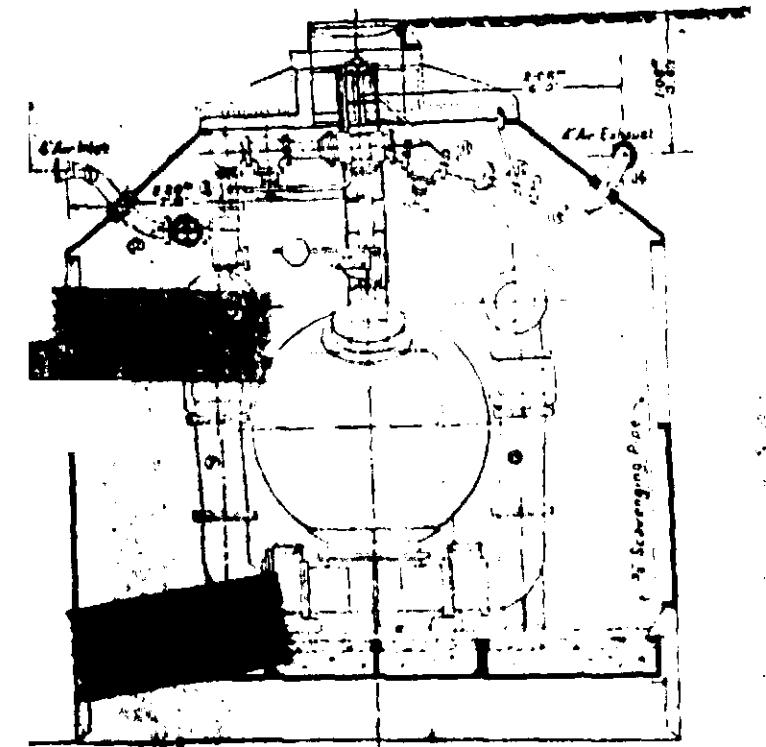
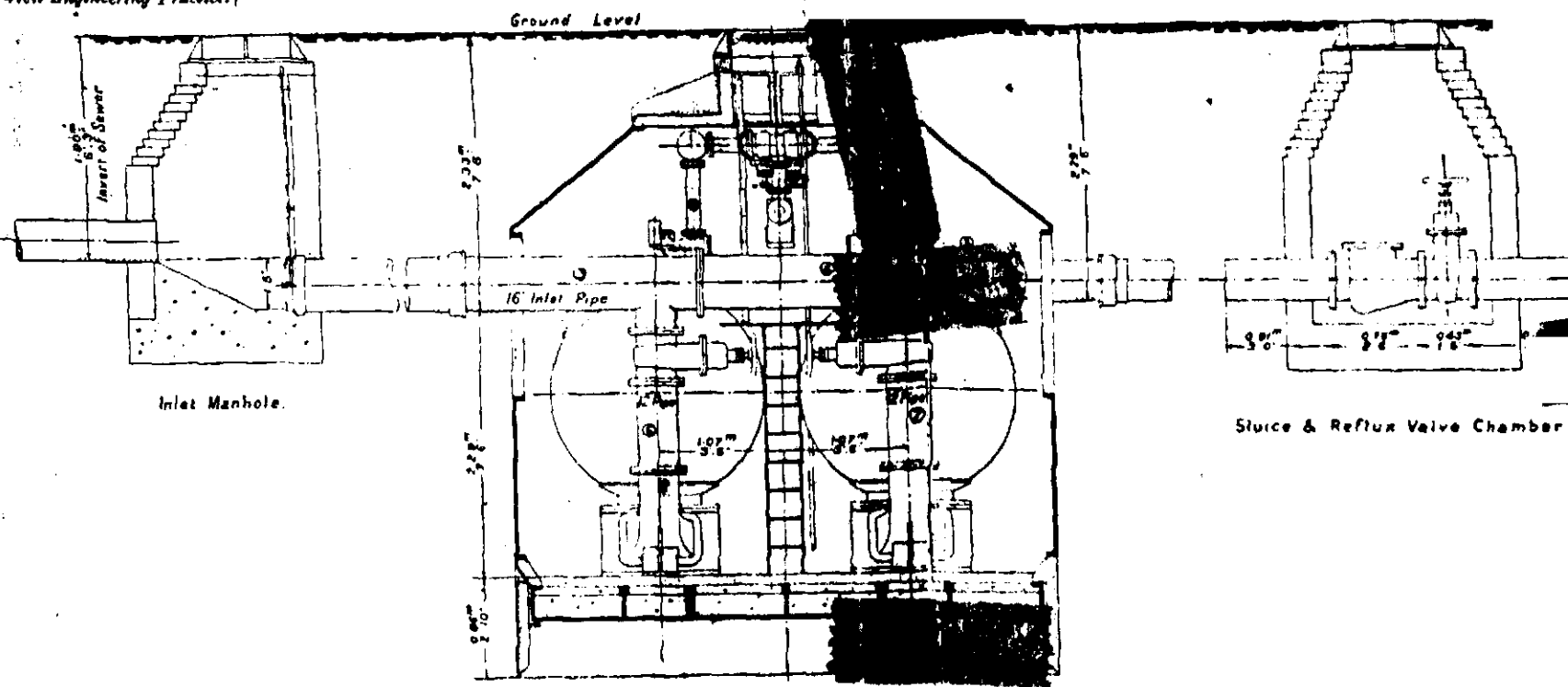


Plan.

With Conical Ring Removed.

FIGS. 592, 593, and 594.

4-18B



Figs. 589, 590, and 591.

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obtained as follows:—Suction head: 13 ft. + 90 ft., rising 1 in 25 = $13 + \frac{90}{25}$
 $= 13 + 3.6 = 16.6$ ft. Friction head,

$$\log h = \log L + 2 \log P - 5.16 \log D = 8.0913987,$$

h being the friction head in the straight pipes, L = length of main in ft.,
 D = pipe diameter in ins. L is made up as follows:—13 + 90 + 18 = 121 ft.

$$\begin{aligned}\log h &= \log 121 + 2 \log 40,000 - 5.16 \log 3.5 = 8.0913987 \\ &= 2.08279 + (2 \times 4.60206) - (5.16 \times 0.54407) = 8.0913987 \\ &= 2.08279 + 9.20412 - 2.8074012 = 8.0913987 \\ &= 0.38811 \quad \therefore h = 2.445 \text{ ft.}\end{aligned}$$

To this we have to add the friction due to the elbows, which is calculated as follows:—

$$h_e = \frac{CV^2}{2g}.$$

C we will set at 0.5 in the present case, as the elbows are probably steam-pipe fittings. The velocity V is determined thus,

$$\frac{p}{3600 \times A}$$

A is the area of the pipe (here 3.5 in. in diameter) in sq. ft. 3 in. is 0.25 ft., and 0.5 in. is one-sixth of 3 in.; therefore $\frac{0.25}{6} = 0.0416$ and 3.5 in. = 0.2916 ft., giving an area of about 0.0668 sq. ft. p is the water quantity per hour, expressed in cub. ft., or 4000 gals. $\times 0.1605 = 642$ cub. ft. per hour. Then

$$V = \frac{642}{3600 \times 0.0668} = \frac{642}{240.48} = 2.66 \text{ ft. per second,}$$

and

$$h_e = \frac{0.5 \times 2.66^2}{2 \times 32} = \frac{0.5 \times 7.0756}{64} = 0.055278 \text{ ft.}$$

There being five knees, we have to take five times the above amount; and as we will allow the equivalent of two knees for the suction strainer, and a similar amount for the regulating or stop valve, we have

$$0.0553 \times 9 = 0.4977 \text{ ft.} = h_e.$$

Adding the previous result = 2.445 ft. = h_i we obtain

$$h_i \text{ (total)} = 2.9427 \text{ ft.}$$

The loss due to outflow is determined as follows:—

$$h_l = \frac{v^2}{2g},$$

or in our case

$$h_l = \frac{2.66^2}{64} = \frac{7.0756}{64} = 0.110556 \text{ ft.}$$

The total efficiency, η , is the result of the combined efficiencies of the pump, the two belt drives, the countershaft, and the gas engine, which may be estimated respectively as follows:—Efficiency of pump = 0.6; efficiency of each belt drive = 0.97; efficiency of countershaft = 0.95; and efficiency of gas engine (if small) = 0.6; therefore $\eta = 0.6 \times 0.97 \times 0.97 \times 0.95 \times 0.6 = 0.3218$. The main formula, therefore, can be resolved as follows:—

$$\text{I.H.P.} = \frac{P = \Pi_{\text{mano.}}}{3600 \times 550 \times \eta} = \frac{40000 \times 37.65}{3600 \times 550 \times 0.3218} = 2.36.$$

Regarding the performances of centrifugal pumps, the following figures are worth the reader's attention. The pumps in question were erected for the Buenos Ayres water supply, and were built by Messrs Gwynne & Co., London.

Regarding the transmission of power by compressed air, the maximum percentage of useful effect between the air compressor and the motor cannot be taken at more than 30 per cent. Assuming a direct-driven air compressor of 85 per cent. efficiency; 75 per cent. efficiency in compressing, 10 per cent. loss per mile on the mains (this is the minimum loss, too, when the mains are of the best construction), and 50 per cent. efficiency of the compressed air motor. Thus if the steam or other motive power develops 100 I.H.P. at one mile from the compressor, we shall have

$$100 \times .85 \times .75 \times .9 \times .5 = 28.69 \text{ H.P.},$$

or if the distance is half a mile,

$$100 \times .85 \times .75 \times .95 \times .5 = 30.28 \text{ H.P.}$$

The velocity of air in the mains must not exceed 40 ft. per second.

The coefficient of friction for the flow of air through wrought and cast-iron pipes varies inversely as the diameter of the pipes, and is obtained by Professor W. C. Unwin's formula as follows:—

$$\text{Coefficient} = .0027 \left(1 + \frac{3}{10d} \right)$$

where d is the diameter of the pipes in ft.

This coefficient is .00377 for $7\frac{1}{2}$ -in. diameter pipes.

" " .00449 " 6 in.

To obtain good results from a compressed-air motor it is advisable, in all cases where possible, to re-heat the compressed air up to 300 or 400° Fahr., at a point near to the motor, the economy being increased, and difficulties with ice, etc., in the motor being removed.

Modern types of steam-driven air compressors will give efficiencies up to 90 per cent., a test on an Alley and Maclellan set giving 89.7 per cent. with compression at 100 lbs. per sq. in. A recent test on a Reidler two-stage compression was carried out by Mr W. D. Hamilton, with the following results:—

	Per cent.
Efficiency of compression	95
" combined engine and compressor	77.8
Percentage loss due to friction of set	9.2
" " in compression	4.5
" " due to clearance, radiation, leaks, and bends	8.5

24 STREAM ADMISSION H.P. CYL.
TWENTY-FOUR HOURS' TRIAL AT 10

Time. Hour.	I.H.P.	Total Head.	Suction head. Including friction.	Delivery head.	Height of water in well.	Height of river.	Water pumped per minute.	"C" Engine Revolutions.		"B" Engine Revolutions.		Per cent.
								Hour.	Minute.	Hour.	Minute.	
8 a.m.	1.46	11.032	6.932	5.35	11.40	11.69	77,000	10.430	179	10.760	179	
9 "	1.44	10.982	6.982	5.60	10.30	11.31	77,000	10.380	182	10.910	182	
10 "	1.329	10.682	5.982	5.60	10.11	11.43	77,000	10.840	183	10.910	183	
11 "	1.772	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
12 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
1 p.m.	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
2 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
3 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
4 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
5 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
6 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
7 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
8 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
9 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
10 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
11 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
12 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
1 a.m.	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
2 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
3 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
4 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
5 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
6 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
7 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
8 "	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	
Mean result	1.701	10.775	5.60	5.60	10.14	11.44	77,000	10.940	183	10.940	183	

4 H.P. CYL.
FULL POWER TRIAL WITH TWO ENGINES, STEAM ADMISSION 10

Time. Hour.	I.H.P.	Total Gross Lift.	Delivery Lift. Gross.	Suction Lift. Gross.	Height of water in well.	Height of river.	Water pumped per minute.	"C" Engine Revolutions.		"B" Engine Revolutions.		Per cent.
								Hour.	Minute.	Hour.	Minute.	
8.30 a.m.	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	
9 "	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	
9.30 "	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	
10 "	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	
10.30 "	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	
Mean	1.067	11.831	5.991	5.840	10.50	12.90	89,650	10.430	179	10.760	179	

Before closing the chapter it may be well to say a few words on Power Stations, or rather Pumping Stations in this case. The design of the pumping station is a matter which might be entrusted to a municipal engineer, and it is important that he should be aware of a few of the essential principles underlying that design, so as to avoid costly and often practically unalterable mistakes. In the first instance, the site is the all-important question. It is of paramount importance to have such a pumping station if possible near to a railway siding, canal side, or dock side, where coal could be purchased and landed with the minimum of trouble and expense. Now a sewage pumping station will more often than not lend itself to such circumstances because of its necessarily being in a low part of the town. In a sea-board town this will be particularly likely to be the case, as seen in the Ringsend pumping station in Dublin, and the Salford and Manchester Corporation stations, both of which have access to the sea for import of coal and disposal of the sludge by the Manchester Ship Canal. Besides, such towns are usually so flat as to warrant a pumping station.

It is important that the engineer should not be too ready to erect a pumping plant on a particular piece of land, just because that piece of land happens to be cheap or otherwise easy of acquisition, or because it actually happens to be a council's property. This latter consideration has unfortunately led engineers into difficulties too often. Again, water, both for feed purposes and condensing, must be readily obtainable. The subsoil, too, is an important factor. Piled foundations are expensive and not always satisfactory, although our previous requirements may very often demand their use. Then there are the smoke and, in the case of sewage works, the smell nuisances to contend with; and although these should not exist to a very great degree in well designed and managed schemes, objections will doubtless be raised if they do exist. Such matters all demand minute attention. It would be a very bad policy to place a station in a district where future extensions would be difficult and costly to provide for. To obtain a site, however, which is going to satisfy all these requirements is far from an easy matter. With gravitation mains, however, a central air-compressing station, by which would be worked a series of Shone's ejectors, within reasonable limits, the station could be placed almost anywhere that conditions of transport of coal and water supply demanded. Now waterworks pumping stations will put quite a different aspect on the matter. The plant may in all probability be in a very out-of-the-way situation, and coal may have to be transported by road. Circulating water will always be plentiful, but the question of feed water may present difficulties if the water pumped, which is very often the case, is of a hard nature; it may need special treatment, as is described in a later chapter. Such considerations (water, coal, and transport of plant in first instance) place before us the vital question of gas or steam engines, to which we shall return.

In the meantime, let us examine the design of the actual buildings.

The main points to observe are—

1. Always provide for possible extension of all parts of the plant.
2. Engines and boilers should always be separate.
3. Feed pumps should not be in boiler-houses.
4. The fuel should be stored as near the boilers as possible, but not in such a position as there would be any risk of fire.
5. A workshop and stores should be provided.
6. Also an engineer's office and, if a large station, a general and clerk's office.
7. A room and bath for the men will no doubt be appreciated.

8. A travelling crane is essential. The building should be designed for a 10-ton load at least.

9. Ornamentation is not necessary on the exterior of power stations.

10. The boiler-house would best be paved with brick on edge, and the engine-house with tessellated paving.

A very important observation is—not to place flues and economisers *below* the ground water-level. Water is sure to leak in, with continual reduction of economy.

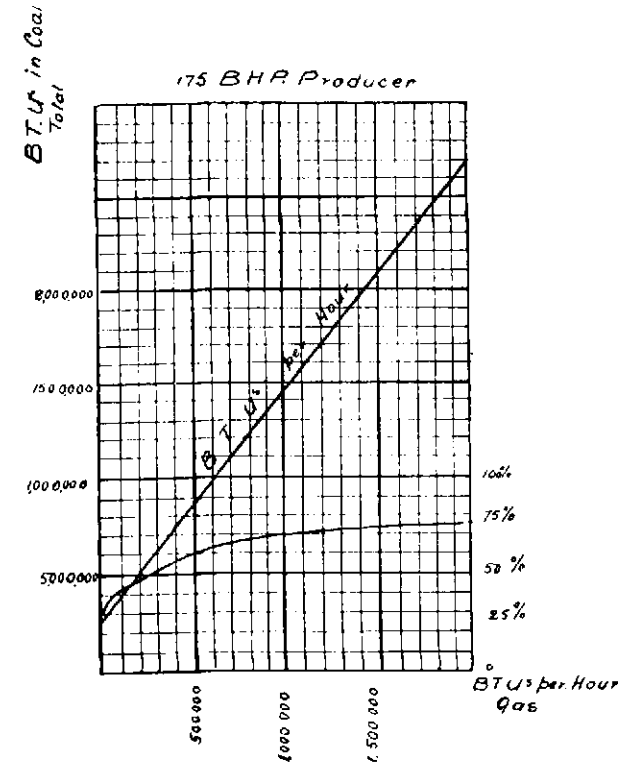


FIG. 594A.

With gas engines the best system of ventilation must be provided. It may have to be mechanical.

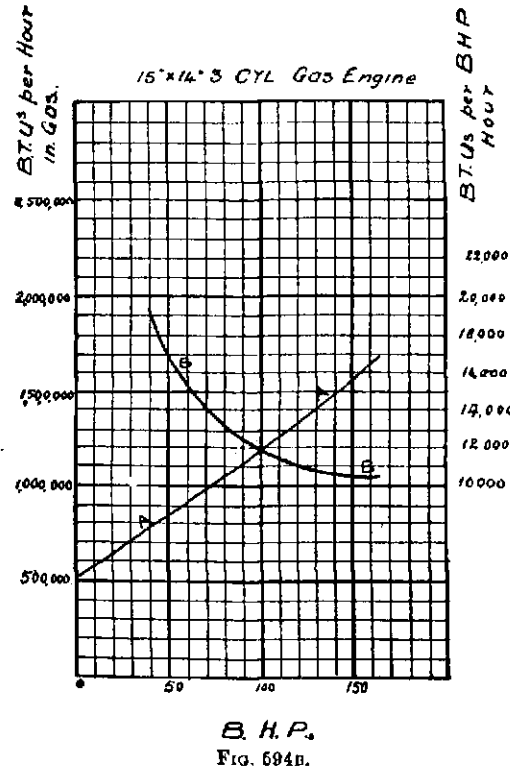
The gas-producer house should, however, always be separate, and the gas holders outside.

As stated, in country districts, for moderate-sized pumping stations for waterworks (or sewage), gas engines demand much attention.

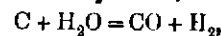
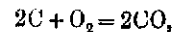
1. The cost of transport of coal will be reduced.
2. With waterwork engines a steady load will always be obtainable, a fact which at once lends itself to economy in working of gas engines, which are not suited to overloads, nor economical at low ones (that is, lower than their rated capacity).

For small plants of any kind gas engines would at once hold the field, but for larger ones it is a debated question.

For the purpose of demonstrating the economy of gas engines the reader is presented with the curves in figs. 594A and 594B, which represent recent gas-engine and producer practice. They are interesting. The engine referred to



passing air and steam through incandescent fuel. Two simple reactions take place, expressed by



which is to say that the oxygen of the air combines with the carbon of the fuel to form carbon monoxide; the water splits up to form more CO and H. In addition, a certain amount of CH_4 and CO_2 are formed, so that the ultimate composition of the gas is combustibles CO , H_2 , and CH_4 diluted with the non-combustibles CO_2 and N_2 . For further information on gas engines the reader is referred to standard text-books.

is a 140 B.H.P. gas engine, 3 cylinders, 15 in. x 14 in., made by the Westinghouse Company. A small engine would give about 10 per cent lower values. Fig. 594A shows some efficiency curves of a gas producer by the same firm. Bituminous coal was used, having a calorific value of 13,000 B.T.U.'s per lb. of coal. It was rated at 170 horse-power.

The practice in Britain at present in small plants is to use suction producers working on anthracite coal, or some form of pressure producer working on similar fuel, like the Dowson producer. In large plants, however, where bituminous slack could be cheaply obtained, and the ammoniacal by-products treated on the process invented by Dr Mond of Northwich, great economy would no doubt result.

Producer gas used in these engines is obtained by

CHAPTER XXV.

THE USE OF WATER POWER.

THAT the use of water power in a scientific and economical manner has not had the attention of English engineers that it should have done is beginning to be felt at last. It is to be regretted, perhaps, that while Englishmen have been devoting their time to the perfection of the steam engine, the foreigner has been bringing the use of water power to a fine art. The lack of energy at home in this direction is due, no doubt, to two things—

1. Not a large supply of water power to be utilised.
2. A plentiful supply of coal for power purposes.

Therefore it is natural that the Continental engineers, and American ones too, have studiously devoted their energies to perfecting turbines. It is surprising how little consideration some engineers give to the subject at all. We may admit that the output required may be greatly in excess of what could be supplied by water, but why not utilise what there is, and help the engines by daily and hourly reducing the coal bill? Turbines find use in a variety of places, electric generation for large supplies and for country-houses being perhaps of primary importance. There is no doubt, however, pumping machinery could well utilise it. A small stream would work a stonebreaker, and the surveyor could have all his stones broken for nothing, saving about 1/4 to 1/6 per ton of road metal.

We are not concerned with old-fashioned water-wheels. They gave, no doubt, to their designers much satisfaction; their efficiency, however, was always low, rarely obtaining more than 50 per cent. Besides, they are too cumbersome, slow, and heavy, and are now universally giving way to suitably designed turbines, which will work on almost any fall available; and when the fall is very high, we resort to a very efficient motor, the pelton wheel. Turbines also have the advantage of very little wear and tear, and the necessity of hardly any attention. The turbine is also remarkably steady in running, especially those of the pressure type; water-wheels are just the opposite.

When there is a running stream there will always be a certain amount of power available.

Say, for instance, the engineer gauges a stream in the manner described under Hydraulics, and finds the average flow is 1000 cub. ft. per minute. Between two convenient points on the stream, 1200 ft. apart, by levelling, he ascertains that there is a difference of surface-level of 10 ft. Then if he places a weir at the upstream point and builds a mill race or head race, viz. a canal having only sufficient fall to allow the water having a gentle velocity of about 1 ft. per second (*vide Hydraulics*), he will have at the point down-

turbines with a high fall, the above arrangement will not hold good; cast-iron piping is invariably used. It may here be stated that the sluice which is shown in fig. 595 is necessary to shut off the water from the turbine in a case of repairs, and *not* for regulating purposes, for which a special apparatus, an integral part of the turbine itself, is used. Small turbines on high or low falls may work in a case of cast or wrought iron which is always under the pressure of the water; while all impulse turbines work in such cases, they, however, not being under pressure, are only of light construction.

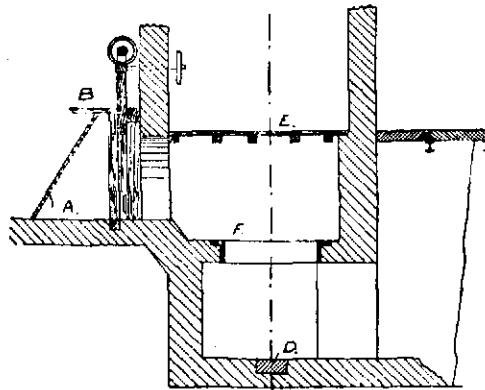


FIG. 595.

We will now place before the reader some typical types of turbine. First of all, we have the inward-flow turbine illustrated in fig. 596. It is shown for a vertical shaft. It may here be stated, the circumstances will generally decide if the machine is to be vertical or horizontal. Needless to say it usually happens a horizontal one is convenient, doing away, as it does, with gearing, and by the use of a draught tube which we shall describe, becomes in most cases quite practicable. When, however, the fall is very low we may

have to use a vertical shaft to get the advantage of every bit of fall, or if the turbine works in a very deep pit it is of course essential. In the inward-flow turbine the water flows in from the outside by the guides and out through the centre. This type is much used, especially in what is termed the mixed-flow turbine, which has the wheel vanes so curved that the water in the wheel is turned downwards through 90° . A very good form of such a machine is that illustrated in section by fig. 597. It is known as the Hercules turbine, and is a good form of low-priced machine where the supply of water is ample and not too much efficiency aimed at. The shaft works on a footstep of lignum vitae, a very hard wood, suited to submerged bearings. The gate is circular, and is raised by a rack-and-pinion motion gear from above. By means of horizontal wings cast upon the vanes it is possible to gain fair economy when the gate is only half open, or what is technically termed "**part gate**." The whole affair (shafting and bolts, etc., excepted) is usually made of cast iron. The wheel may with advantage be made of bronze. It would work in a wheel pit similar to what we have described, or cast-iron or steel casing. Then we have the outward-flow turbine shown in fig 598, in which the action of the water is reversed, and is not so favourite a machine as the inward-flow, because the wheel and moving parts are very ponderous, and the centrifugal effect of the water at high speeds causes an increase of speed by accelerating the flow of water, whereas the opposite should take place. It is unsteady in action. For

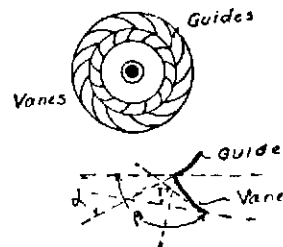


FIG. 596.

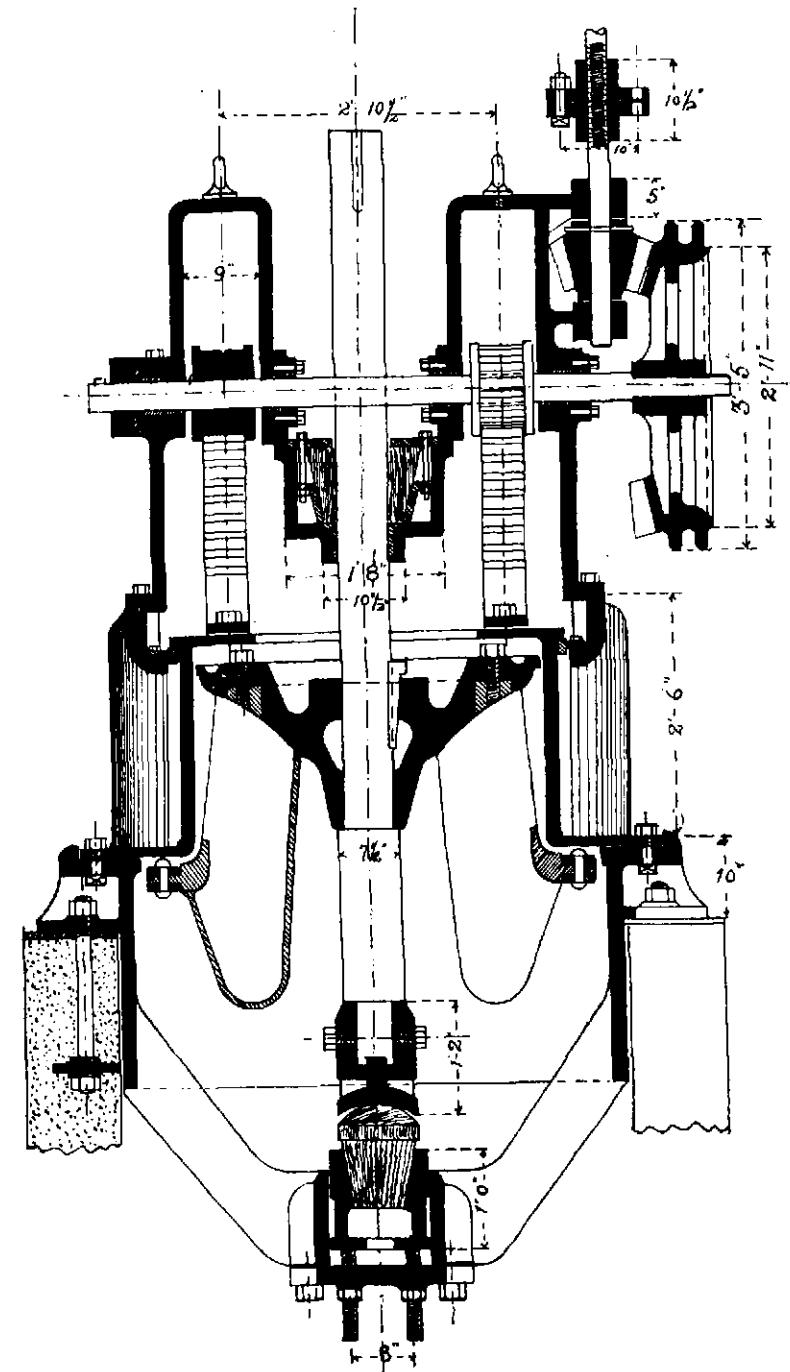


FIG. 597.

stream 1000 cub. ft. of water falling 10 ft. = $1000 \times 62.5 \times 10 = 625,000$
 foot-lbs. of energy per minute = $\frac{625000}{33000} = 18.9$ theoretical horse-power; and
 if a turbine of good design is installed which will easily give 80% efficiency,
 he has

$$\frac{18.9 \times 80}{100} = 15 \text{ available horse-power.}$$

Of course larger and swifter streams give much larger results. We only give the example to show what can be done.

We said before that we are not going to advise any engineer to instal a water-wheel; at the same time a few words in reference to them will not be out of place. First of all there is the over-shot wheel. It consists of a wooden or iron wheel which has a number of blades, which, together with the inner circumference, form a series of buckets. Into these buckets the water which is led to the top of the wheel by a pentrough (usually of wood supported on steel joists) falls and acts by gravity alone. In the pentrough is the regulating sluice. The buckets are designed to hold the water as long as possible so as to use as much of the power as they can. The inherent velocity of the water assists the process. It is not an efficient motor.

Then there is the breast wheel. There the wheel runs in a sort of masonry casing or "breasting" with very little clearance, the water being admitted about halfway up the wheel. The old breast wheels had ordinary flat radial blades or paddles, and acted by impulse only. A more modern form, much seen at the present day, is Fairbairn's wheel, with curved steel blades, in which the water acted to a great extent by gravity. The buckets usually have no ends, the breasting having a fine clearance. They must, however, be ventilated. Then there is the undershot wheel. Here the water enters by a sliding hatch on the lower circumference of the wheel, and acts by impulse or momentum. A later form, however, has curved blades, introduced by a French engineer, Monsieur Poncelet. The introduction of these blades was at the time considered a great improvement, and the efficiency went up by leaps and bounds to nearly double that of the old wheels. There is no doubt also that Sir William Fairbairn did much to advance the water-wheel, and many are still giving satisfactory results where efficiency has no weight. But there is no doubt that the modern hydraulic motor is the turbine.

We are not going to deal with hydraulic power as applied to cranes, dock machinery, etc., under very high pressures generated by artificial means—it would be out of the scope of the present work; but to those interested I would recommend Mr Croydon Mark's book on *Hydraulic Power Engineering*. What we have to deal with are natural sources of power. Now the first thing the engineer has to do is to find out if he has water power available. If he has, he will have to find out the total available fall, being the difference between the head and tail water-levels when water is passing.

Then the quantity must be carefully gauged.

He must then consider if it will be economical to store some of the water. For instance, in a small electric generating station the load will only be required for, say, a maximum of 8 hours per day. Then, if the day flow is stored up, he would have something like three times the available energy. But the question arises, Will the outlay warrant the procedure?—another opportunity for his practical experience. Then he will have to consider how the power is to be conveyed. For low falls, the head race or canal constructed

practically level as in fig. 498, the water being diverted and stored up by a weir, is the most usual procedure. Or a circular shaft in mid-stream and a tunnel driven to the turbine may be deemed advisable, although an expensive procedure. For high falls and utilising water from lakes, a pipe line of cast iron or riveted steel is generally adopted. Then he has to consider what horse-power he will require, and if, as is often the case, the new turbine is to replace an old, inefficient water-wheel, what horse-power the wheel gave out, and if the position of the wheel pit is suited to the turbine. Fortunately, turbines are compact machines, and it is usually a matter of no great difficulty to place a turbine in the wheel pit. Then he must decide if a vertical or horizontal spindle is to give the best results; and finally, the direction his turbine is to run in. These particulars are usually required by the manufacturers. Then the first thing to do is to construct the **Head Race**.

In constructing the head race a very frequent error is committed by failing to give it sufficient capacity. It should be **wide and deep**, and especially where the race is of a considerable length. As a general rule, the water should not flow faster than from 60 to 120 ft. per minute.

When the water is conveyed to the turbines through iron pipes, the receiving ends should be well submerged, so as to prevent any possibility of their drawing air. The velocity of water through pipes should never be excessive, and should never exceed 4 ft. per second for short pipes; and as the length of pipes increases, the velocity should be reduced. It is, however, often necessary to run the water at a greater velocity than above named.

Secondly, he must design the **Tail Race**. This also should be **wide and deep**, and the level of the bottom of the wheel pit should be carried out from 5 to 10 ft. below the end of forebay or flume, and if possible it should be carried out to the bed of the stream. By having the tail race thus constructed, as soon as the water is discharged from the wheel or wheels it will push out or displace the dead water in the race, thus preventing a loss of head.

For instance, suppose the bottom of tail race is on a level with the water in main stream into which tail race discharges, when wheels are started the water in tail race would have to rise in proportion to the width of race and quantity of water flowing therein, and reduce the working head in proportion; while if the race was as first above stated, the water from the wheels would displace the water without rising above the water in the main stream, thus utilising the full amount of head.

And thirdly, the wheel pit. In putting in turbine wheels, whether under high or low heads, the pit must be wide and deep. This is most important where a large wheel is run inside a low head, as under such circumstances every available inch of head must be used. A pit of insufficient size causes the water to react upon the wheel. An additional loss of power is also caused by the fact that a portion of the head is consumed in forcing the water out of the pit when there is not sufficient outlet.

A suitably constructed wheel pit for a turbine having a vertical shaft is shown by fig. 595. It is adapted to a low-fall turbine and has an open flume. The walls would be built of concrete or rubble masonry set in cement mortar. At A is a strainer of the usual pattern made of $\frac{3}{4}$ in. round bars in a frame. It is removable. At B is a wooden sluice, which may be similar to pattern in fig. 490 or fig. 491. It is worked by a hand wheel. At D is a special block of masonry to which the shaft is bolted, while F is a cast-iron ring for a similar purpose. Then there is the ordinary wooden floor at E, through which the turbine shaft passes, and on which will be placed all the regulating gear. For

low falls we have the parallel-flow turbine, fig. 599, in which the guides are placed above the wheel. The Jonval turbine works upon this principle. It is the most efficient parallel-flow turbine on the market, but is not so efficient as a good Girard machine, especially at part gate. The regulation of this turbine consists of a slide which closes the inlet parts one after the other. By

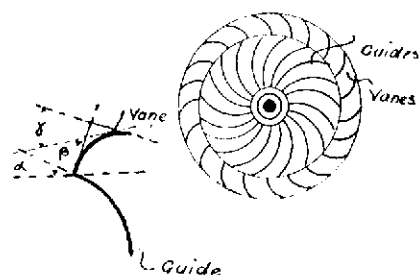


FIG. 598.

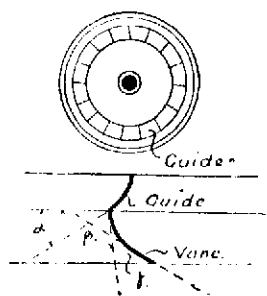


FIG. 599.

this method the angle at which the water enters the wheel is always the same, a most desirable qualification.

It will work well when immersed in tail water or by a suction tube. The action of a suction tube is that of atmospheric pressure being able to hold up a column of water to 34 ft. high (*in theory*). This enables a turbine of the pressure or drowned type to work when placed any height up to 25 ft. above tail water without loss of head, provided the connection between the turbine and tail water be by means of an air-tight tube, whose end is immersed totally in the tail race. An inward-flow turbine with a horizontal shaft is shown coupled to such a suction or draft tube in fig. 600. Now a type of turbine which has perhaps found as much favour as any for general purposes is the Girard turbine. It is what is called an impulse turbine, viz. the water issues from the guide ports with the full velocity due to the head, and glides along the concave surfaces of the vanes without touching the outer ones. The machine is efficient at nearly all speeds. The

machine may be vertical or horizontal. The wheel is shown in two views in figs. 601 and 602, in which will be seen ventilating apertures, which are essential for the proper working of the turbine, so that the water will pass over the wheel vanes in perfectly unbroken streams. Small angles of discharge on the vanes are also essential, but at the same time we must have an ample out-flow area. This is provided in the widened bell-mouth shape of the buckets.

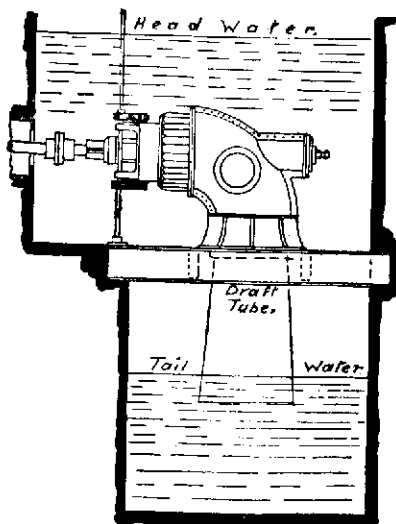


FIG. 600.

The machine will not work with a draught tube. It must be placed 2 in. to 3 in. above tail water, but when slightly drowned the efficiency does not appreciably fall off. The adjustment is effected by opening a series of guide ports in succession. This does not alter the angle of admission, neither do the guides extend right round the turbine like those of the pressure type, except under exceptional circumstances for low falls. We have shown the types of buckets adapted to a vertical machine. But the turbine can be easily made with a horizontal shaft, in which case the admission must be partial. They are

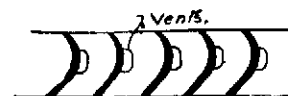


FIG. 601.



FIG. 602.

used on higher falls. At the same time it may here be stated that the Jonval turbine is not confined to vertical shafts; machines placed in suitable cast-iron cases have given very good results. The guides and vanes of a horizontal Girard appear in fig. 603, and those of a horizontal Jonval in fig. 604. Now another turbine of the pressure type which we cannot overlook is the vortex turbine, designed by Professor James Thompson. It is applicable, like the Girard machine, to falls from 10-200 ft., and has given very satisfactory results.

An illustration of such a turbine appears in fig. 605, where great similarity to a centrifugal pump will be observed to outward appearance. The water enters the machine at the top and flows past specially formed guides, usually numbering four or more on to the outer circumference of the revolving wheel. The water having expended its energy, is discharged through two central openings, usually connected to two suction pipes. The speciality of the invention is the special guide blades. These, curved in form, turn about their inner ends on pivots. They are all connected mechanically by levers moved simultaneously by a hand-wheel. The water, due to the

special design, works half by fluid pressure and half by its velocity, thereby nullifying the effects of any centrifugal force. The reduction of water velocity is also an advantage. Being also of the inward-flow type, it has all the advantages set forth regarding that form. There is also no loss from shock at entry, because the wheel is travelling at half the velocity due to the head. It is also efficient at part gate, which is a distinct point in favour of the machine; because the channels are of a gradually convergent form, the water is uninterrupted in its course, and it enters around the whole circumference of the wheel. In this turbine there is no end thrust, and it may have a vertical shaft, but usually has a horizontal one, as the turbine, though it has proved efficient on low falls, is more adapted for

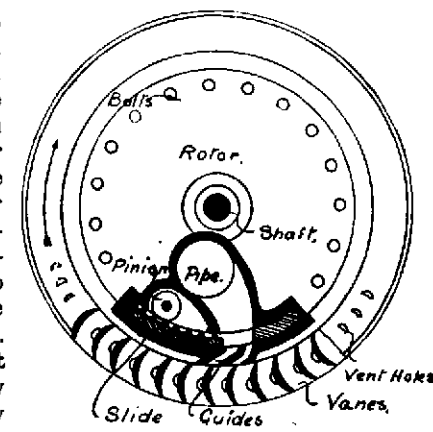


FIG. 603.

falls of 50 ft. and upwards. The case is usually of cast iron, but it may be built up of steel; while large-size wheels are of steel, the smaller ones are of cast or rolled bronze. Like what we said in reference to centrifugal pumps, air must be kept out of the stuffing boxes, without screwing them up so tight as to cause friction. This is effected by small copper pipes attached to the pressure pipe, which always keeps them water-sealed. These turbines are built in the double form also, one wheel at each end of a cast-iron or steel case; each wheel has then only one discharge pipe; in fact, most turbines on a horizontal shaft

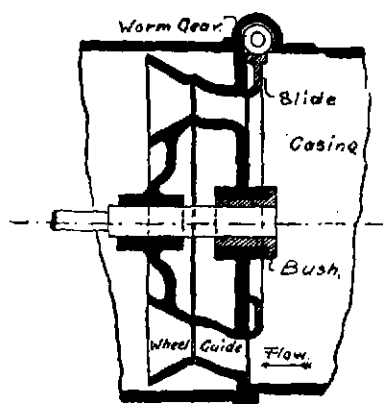


FIG. 604.

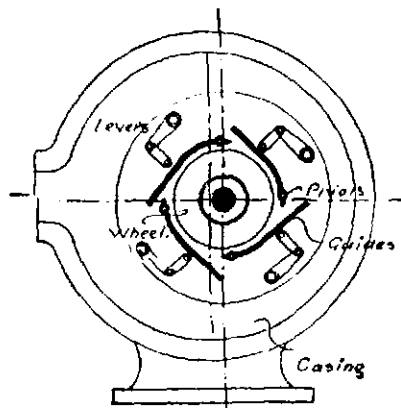


FIG. 605.

are made in pairs; it avoids any end thrust that would involve the use of a special thrust block, which, no matter how good the design is, are well avoided.

Before going any further, we will present to the reader a few words bearing on the theoretical aspect of water turbines, which in many ways resembles that of the centrifugal pump.

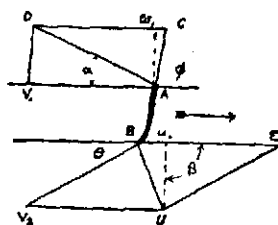


FIG. 606.

Take the diagram in fig. 606. Here we have—
AB = the vane moving in the direction of the arrow.

AO = absolute velocity of the water = V .

OV₁ = relative velocity of the water to the vane = V_1 .

OC = peripheral speed of the vane = C_1 .

Ow₁ = velocity of whirl at entry = w_1 and is perpendicular to OC.

Bw₂ = velocity of whirl at discharge = w_2 .

BV₂ = relative velocity at discharge = V_2 .

BE = velocity of vane = C_1 .

The angle Bw₂u should be a right angle.

Now after AO and AC have been plotted and also the correct profile of the vane drawn, if CA is not tangent to the vane at A, the water will not enter without shock as it should do, and the vane has to be altered in design. Again, naturally the velocity at discharge should be as small as possible so as to give up all the available energy to the wheel we can, because the loss of energy is equal to

$$u = \frac{w^2}{2g} \quad (376)$$

The maximum efficiency is obtained when the angle Bw₂u is a right angle.

The hydraulic efficiency

$$= \frac{C_1 w_1}{gh} \quad (377)$$

The loss of head due to friction in the guide passages

$$L = \frac{FV^2}{2g} \quad (378)$$

F being equal to .125. The losses in the wheel are practically the same, but in this case the value of F = .210. Now the work done by every pound of water which passes the wheel

$$= \frac{C_1 w_1}{g} \quad (379)$$

The velocities in the diagram refer to the mean radius of the wheel, and apply equally well to axial as well as radial, inward, and outward flow. It was before stated that the water enters a turbine wheel with a certain velocity and leaves with a considerably lower velocity, the energy given up to the wheel being equal to the difference between these two velocities, which necessitates the velocity at outflow being as small as possible, the initial velocity being of course determined by the available head. It would perhaps appear obvious to the reader that the area of the guide passage would be equal to

$$A = \frac{Q}{V} \quad (380)$$

But allowance must be made for the vanes, which themselves will occupy some space, and we alter the quantity to

$$A_1 = \frac{Q}{.89V} \quad (381)$$

so by choosing a suitable radius the width of the vane can be settled. We also have the small head = h_0 fig. 607, to deal with in parallel flow (Jonval) turbines with a vertical shaft. With high falls it is of course unimportant, but must receive every consideration on low falls. Again, like outward-flow turbines, in axial turbines (whether Jonval or Girard types) there will be a certain centrifugal force, to avoid which (or at least to counteract it) the shape of the vane is altered to the form shown in fig. 608, shown by the full lines. When we have decided upon r and settled A_1 we find the values e_1 and e_2 by the following equations,

$$e_1 = \frac{A_1}{2\pi r \cos \alpha - (z_1 t_1)} \quad (382)$$

and

$$e_2 = \frac{A_2}{2\pi r_2 \cos \beta - z_2 t_2} \quad (383)$$

A_1 being found by Equation 381, t_1 being the thickness and z_1 the number of guide vanes, and t_2 and z_2 referring in the same way to the wheel vane, A_1 being the area necessary to pass off the water at the velocity of discharge, and as this must be very small, we have still another reason for the bell-mouth buckets. z_2 should always be slightly (1 or 2) less than z_1 , and for the same

reason the width of the vanes is slightly increased above the theoretical requirements. Dealing with reaction turbines—the above reasoning referring to impulse turbines—we can design the vanes as before, making $A_1 = A_2$, because of the law of continuity, $a_1 v_1 = a_2 v_2$, etc. To plot the diagram following we require to know V . If then we settle A , A_1 , and A_2 , fig. 609, the formula to find

$$V = \sqrt{2gh} \sqrt{1 + \left(\frac{A}{A_2}\right)^2 - \left(\frac{A}{A_1}\right)^2} \quad (384)$$

This formula cannot, however, be used direct as it stands because, although we may assume A and A_2 , A_1 is dependent entirely upon their values and that of α and β (vide diagram 606), so that the values have to be found when



FIG. 607.

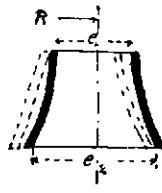


FIG. 608.



FIG. 609.

the formulæ are used for radial wheels instead of axial wheels: C_2 , instead of being equal to C_1 , is obviously equal to

$$C_1 \frac{r_1}{r_2} \quad (385)$$

r_1 being the radius of the vanes and r_2 of the buckets. The loss by leakage, then, between the wheel and guides may be taken as equal to 4-5% of the total head. The depth of the buckets is usually 8 in. to 12 in., and the guide vanes are made of depth $= \frac{3}{4}$ to 1.0 of the buckets. Now in reference to what we said before concerning the suction tube, and also our reasoning in Hydraulics, we do not get a solid stream of water in the tube, as may be at first supposed, but the flow will take place as shown in fig. 610, in which we have in the centre of the tube a core of comparatively still water. To



FIG. 610.

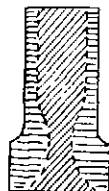


FIG. 611.

nullify the effects of this a draught tube must always be bell-mouthed. It is also an advantage to make them as illustrated in fig. 611, having a plate at the bottom and a radial outflow.

The remarks about the final velocity at outflow will of course apply to all turbines of the pressure type, whether radial or axial flow.

Regarding the vanes of turbines, the smaller wheels are on high-class machines made of bronze (cast), while others are made of cast iron. All bronze wheels should be polished, those of cast iron finished in a sand blast. It is the practice of American engineers to use cast iron very extensively on wheels of all sizes. It is no doubt a cheap practice, but it is questionable

whether it is policy for very large wheels. Wrought-iron or steel blades seem to give better results; at least flaws can be detected more easily and the surfaces can get a better finish, and any breakage does not once for all ruin an expensive casting. At the same time it is better to withstand high stresses due to centrifugal effect in high-speed turbines; in any case a cast-iron rotor must have steel rings shrunk on to guard against bursting. When of wrought iron the blades are usually $\frac{1}{4}$ - $\frac{3}{8}$ in. thick, and if of cast iron $\frac{1}{2}$ - $\frac{3}{4}$ in. thick, there being a few less blades in the rotor than the guides.

Of course the greater the value of r (=radius) of any wheel the smaller will be the widths of wheel and guide vanes; at the same time the wheel is liable to become abnormally heavy; a happy medium must be aimed at. Further, the depth of the wheel when great has the advantage of allowing a more gradual change of direction, but also adds considerable weight. If we know the pressure at inflow $= P_1$ and the pressure at outflow $= P_2$ and $W = 62.5$ lbs., we have ϵ = degree of reaction

$$= \frac{P_1 - P_2}{WH} \quad (386)$$

H being the total head in feet. ϵ may be considered equal to .5 in most cases, and when such is the case we shall have the maximum mechanical efficiency, which causes V to be equal to $\sqrt{gH}$ and the pressure in the clearance space

$$= .5WH \quad (386a)$$

The efficiency will fall when the speed at the outer radius exceeds $\sqrt{2gH}$. In very high-powered turbines of the Jonval type it is usual to divide the machine into three rings, and shut off one ring as required when less power is wanted. The wheel of such a turbine is shown in fig. 612.

With impulse turbines of the Girard type it is usual to make both α and θ small, say 15° - 18° for α and 13° - 16° for θ , but to prevent large-power machines getting very heavy in size these values are usually increased. The essentials of good turbine design may be summed up as follows:—

1. The fluid must enter the wheel either under pressure or with a velocity relative to the wheel and without shock.
2. It should leave with as little velocity as possible, and this velocity must be along the turbine blade.
3. The wheel must run steadily, at a uniform speed.
4. It must not vibrate, and hunting must not occur.
5. Lightness of moving parts and strength are essential.
6. Efficient regulation must be provided, and the machine must give satisfactory results at low powers. The blades of turbines are designed as follows:—

Case I., fig. 613.—Blades of an inward-flow turbine (Molesworth). Lay off CB and CE to scale = inner and outer radii, make the angle $BCE = \gamma - \beta + 180^\circ$. Join EB , then the point A where EB cuts the inner circle will be the inner end of the vane. Lay off $CBD = \beta$, bisect AB and erect a perpendicular cutting, the line BD at the point O , which will be the centre of the vane curve.

Case II., outward flow, fig. 614.—The same construction applies here, only



FIG. 612.

kindly supplied to me by Mr J. Orten Boving, M.I.M.E., of 72 Victoria Street, Westminster. Such vanes in ordinary cases would be moved by hand, but on electrical plants a special oil-pressure governor, actuated by a force pump driven off the main shaft, is used. Then there is the scroll regulator. It consists of a circular scroll, very like a venetian blind, upon which is fixed a strip of leather or rubber attached to small strips of wood or bronze; two rollers, actuated by gearing from a hand-wheel, roll up the scroll around themselves, and so set the turbine in motion. The mechanism, being delicate, is very liable to get out of order. The same method of regulation as shown by fig. 604 is applicable to inward-flow vertical-shaft turbines by means of a

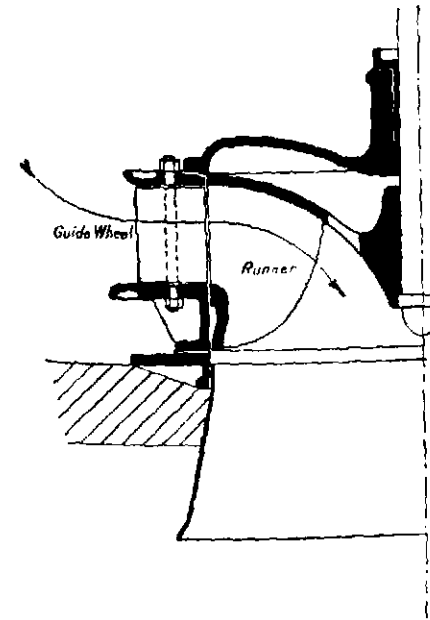


FIG. 617.

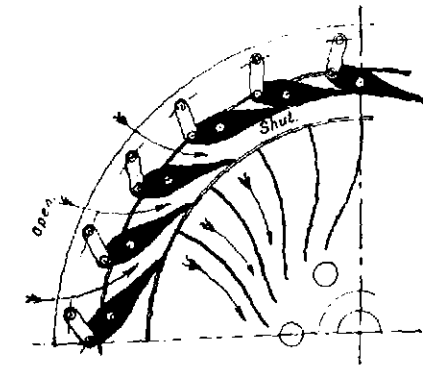
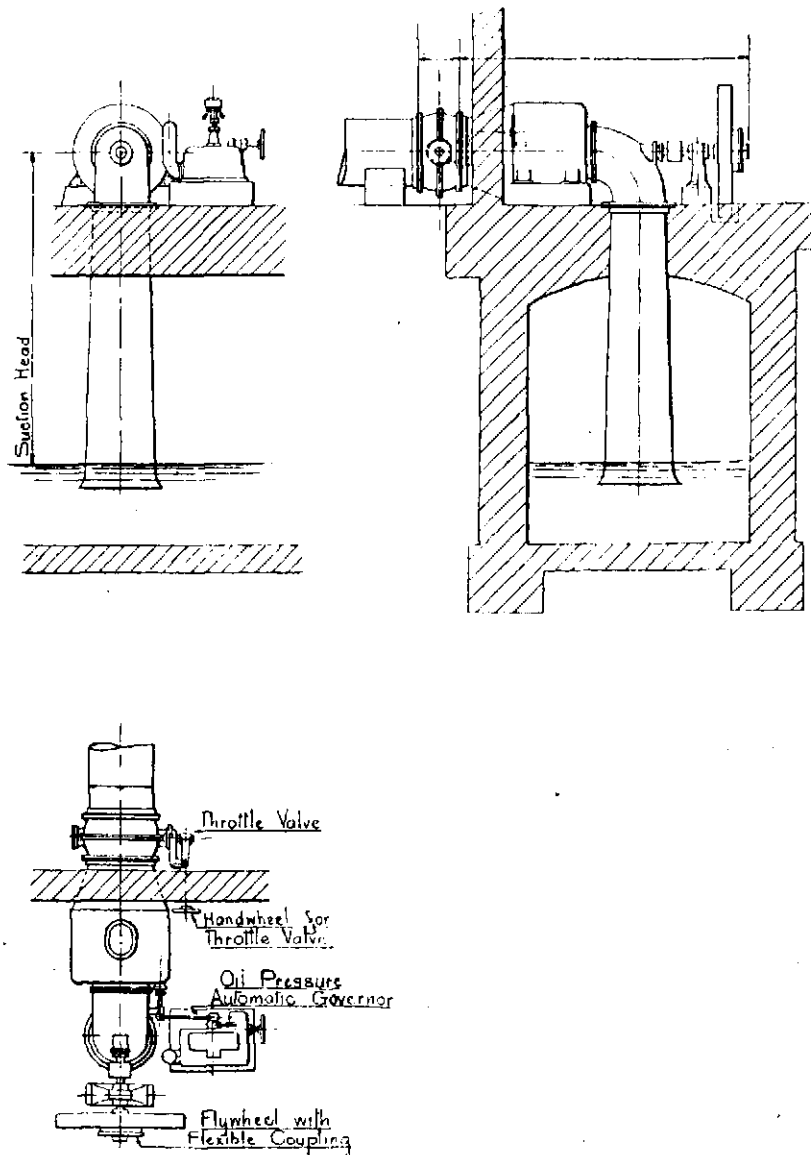


FIG. 618.

circular ring outside the guides, the guide passages on one side of the turbine being higher than those on the opposite side.

The two half-rings are able to slide off their respective passages without affecting the other. As an example of a very large-power turbine having a vertical shaft, we give figs. 619 and 620. It is of a type similar to those running at Niagara Falls power station, U.S.A., and is shown in full detail. There are two wheels, and each wheel, for the purpose of efficient regulation, is divided by diaphragms into 3 circular rings; the slides shown, when moved by hand or by governor, close or open the passages as required. Figs. 621-2-3 show a turbine for falls varying from 30 to 120 ft. It is a form well adapted to direct-drive, and runs at a fairly high speed. For electrical work a flexible coupling must be provided. The actual turbine of the Francis type will develop 100 B.H.P. under 33-ft. head, running at 500 R.P.M. The regulation is illustrated, 617 and 618. Altogether the design (by Mr J. Orten Boving) is worthy of attention.



FIGS. 621, 622, and 623.

The Francis turbine shown in drawings Nos. 624-5-6 is of a type used for low heads of from 4 to 15 ft., the machine being placed in an open flume. On account of the low speed an increase of about 1:3 is obtained by means of bevel gear, and by this means a direct-drive from the horizontal shaft can, as a rule, be used.

Drawings Nos. 627-8-9 show a horizontal Francis turbine suitable for heads of from 10 to 30 ft. When the head approaches the latter figure a considerable portion can, if desired, be taken up in suction head, thus obviating the necessity for a very deep pit.

Fig. 640 shows a good design of impulse turbine direct-coupled to an

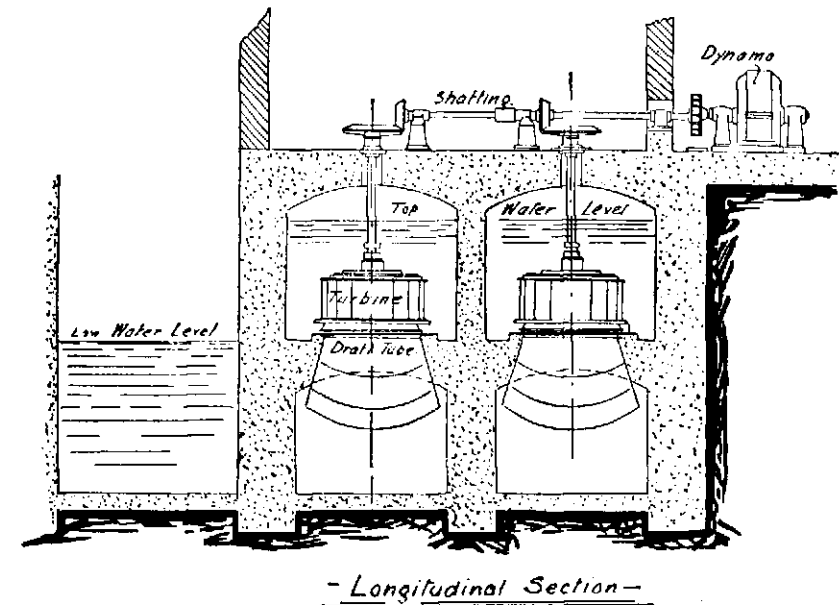


FIG. 624.

electric generator, with a direct-acting governor driven by gearing from the main shaft.

As an example of what could be done with a small turbine installation we have figs. 630 and 631. It is fully described, and was specially designed by the author for a small workshop. It will be seen that owing to the low fall it was only necessary to place the turbine on a timber head race. The plant as erected gave every satisfaction. We have now to describe one more type of water motor, which is so simple in manufacture and construction, and yet when well designed will give such excellent results. It is known as the pelton wheel, and is illustrated in figs. 632 and 633. It consists of a plain disc, mounted on a wood or cast-iron frame, having a number of curved buckets placed at equal distances apart on the periphery; a conical nozzle is attached to the supply pipe, and directs a jet of water on to each of these in turn, while the buckets have a central division (or knife edge) to deflect it into two portions, outwards and backwards. There is no doubt that for high falls, and in out-of-the-way places, for driving every class of machinery, the

instead of CB being equal to β it is made equal to γ (fig. 614). For turbines of the parallel flow type we have the following construction, see figs. 615 and 616.

For guides, fig. 615, set AB = pitch of vane, make $CAB = \alpha$ (fig. 606). Draw BCD at right angles to AC and produce to D , and with centre D describe the arc CE . Then AC and BE being parallel there is no contraction. $\therefore K$ = coefficient of contraction = unity.

The wheel vanes are described as follows (fig. 616): set AB = pitch and $CAB = \theta$ (vide fig. 606).

The curve has now to be so designed that the angle $GEF = \phi$. To do so

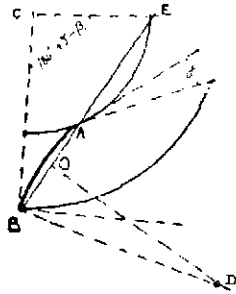


FIG. 613.

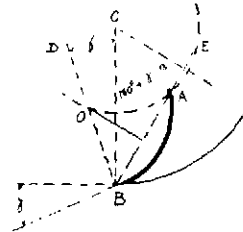


FIG. 614.

take any line HL , making $GHL = \phi$. Draw BCD at right angles to AC , and make $KHL =$ a right angle. Bisect angle K , and a line CE at right angles to the bisector gives us the chord of the arc. The centre D will then be found by trial and error. Now the method by which turbines are regulated is an important consideration, especially where the power available is not in any way too plentiful. We have already described the method applied to

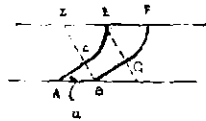


FIG. 615.

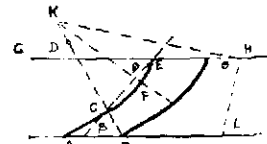
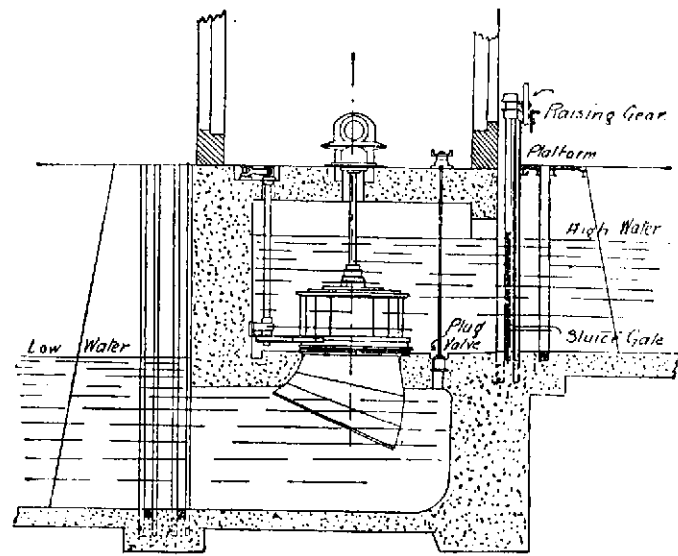


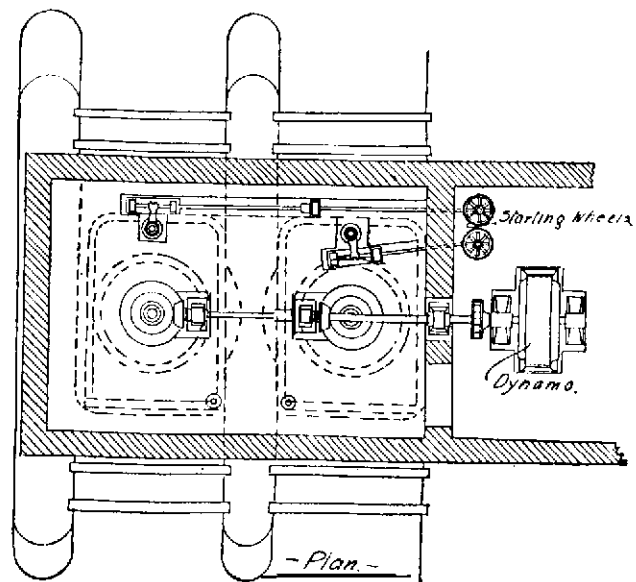
FIG. 616.

Thompson's vortex turbine, and also the method of regulating the Girard turbine and the Jonval turbine. All regulating gear, to be of any advantage, should aim at closing a few passages **totally** instead of all passages partially, and that is now the trend of modern practice. A sluice gate on the head race is quite unsuitable; in the same way, throttling the suction tube is not much better. The sliding gate applied to the Hercules turbine is more or less satisfactory, as the type of turbine is not a particularly high-class machine, although well adapted to the purpose of a low-priced turbine for ordinary purposes. But nowadays much electric generation is done by turbines, and the regulation becomes important. A form of regulator which has given much success is that shown by figs. 617 and 618. It is the principle of movable guide blades, and is applied to the turbine illustrated in fig. 621. The drawings were



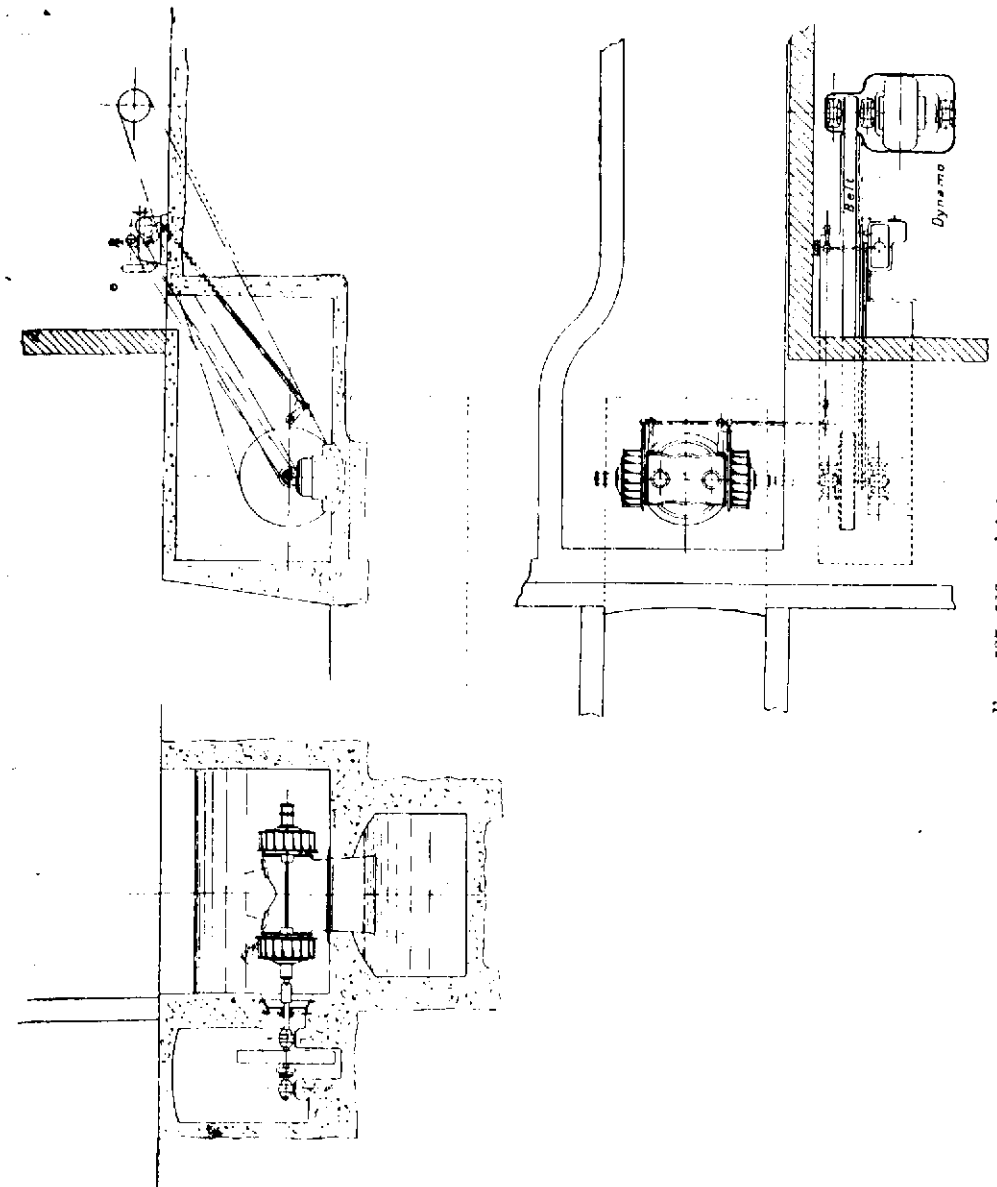
- Cross Section -

FIG. 625.



- Plan -

FIG. 626.



FIGS. 627, 628, and 629.

while the free velocity of the buckets will be

$$4\sqrt{h} \quad (389)$$

The theory of the wheel is as follows:—

Say we want to find the efficiency. We know V = velocity at the nozzle

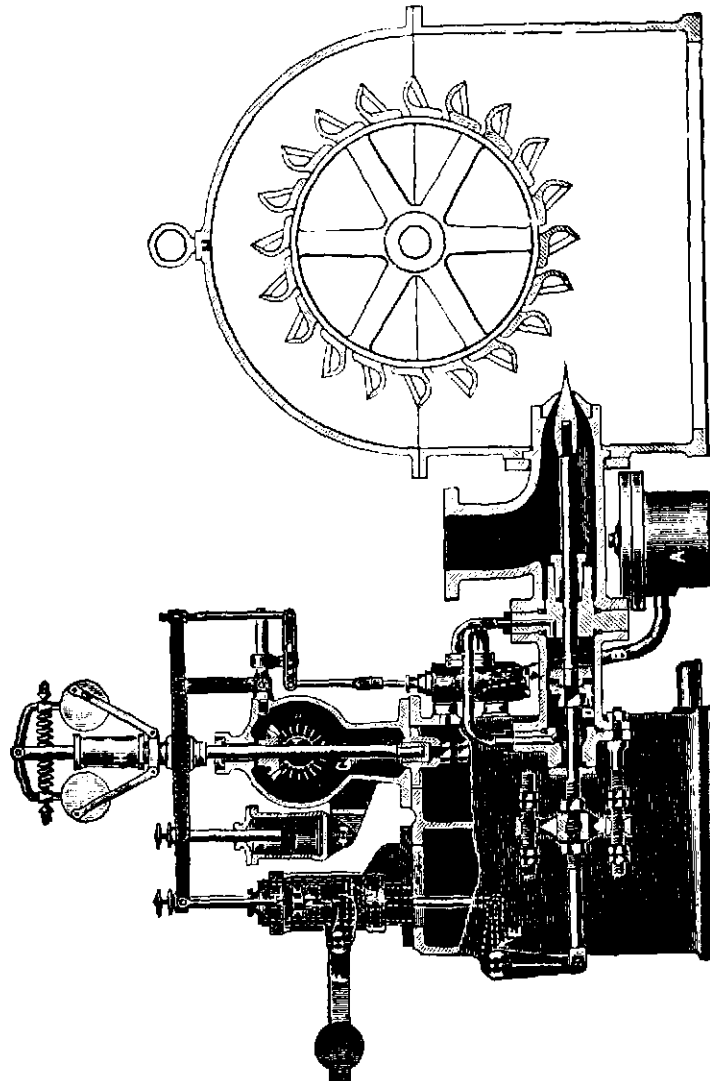


FIG. 632.

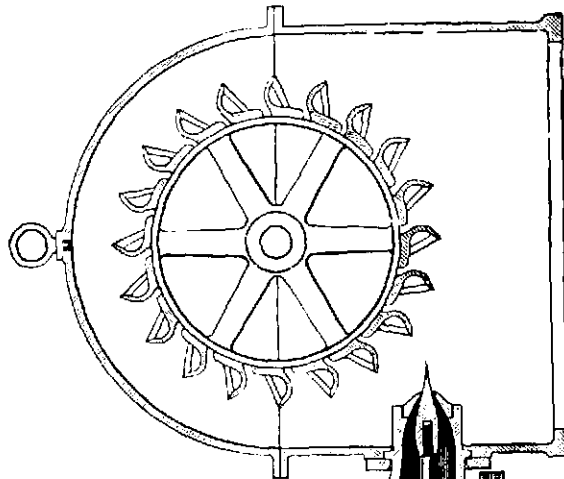


FIG. 633.

of the jet $= 8\sqrt{h}$, less any pipe friction (*vide* Hydraulics). Then if v = velocity of vane in feet per second,

$$v - (V - v) = \text{rel. velocity} \quad (390)$$

The final velocity

$$= 2v - V \quad (391)$$

And the efficiency

$$= \frac{V^2 - (2v - V)^2}{V^2} \quad (392)$$

or as it can be modified into

$$\frac{4v(V - v)}{V^2} \quad (393)$$

This is the maximum efficiency, and is greatest when $V = 2v$. It assumes, however, that the back flow shall be parallel to the forward flow as in fig. 634, but in practice this is not so, as it would collide with the on-coming vane. To find the final velocity in fig. 635, AC = velocity of wheel. AB = velocity of water from jet. AD = final velocity, and energy carried away $= AD^2$. Owing to the fact that the vanes travel very fast, the water strikes two vanes at a time. Some wheels have a double set of buckets.

We before stated that on large turbines and pelton wheels the regulating gear, besides being capable of hand regulation, must also have a sensitive governor. There are many types to be had, but for simplicity and general good design we have taken the one illustrated in figs. 636 and 637.

The hydraulic cylinder rests on planed seatings, cast solid with the slide, over which the crosshead travels. The coupling-rods between the latter and the deflectors are always in tension, so that the crosshead is always pressed downwards when the deflectors are in the path of the jet. The hydraulic cylinder derives its supply of pressure water direct from the pipe-line through the valve-chest shown in position at F, fig. 636, and separately, fig. 637. The inlet to this chest is at A, the entering water passing first through a strainer and then through a somewhat small nozzle, intended to ensure smoothness and regularity in the motion of the relay piston. The exhaust port is at B, and from D and C pipes lead, as shown, to opposite ends of the relay cylinder. The admission of the pressure water to this cylinder is controlled by a small piston valve of phosphor bronze, marked E in the drawing. The valve-chest is itself of Delta metal.

The position of the valve relatively to the governor balls is adjustable by means of a screwed link and the lock-nut shown at I. The governor is operated by a belt drive and bevel gearing as indicated.

At normal speed, water is shut off from both ends of the relay cylinder, but a change of speed in one direction or the other is followed by admitting water to the corresponding end of the relay cylinder, and thus moving the deflector in or out of the path of the jet. Under reasonable conditions of work the governor will maintain the speed constant within a limiting variation of 3 per cent., and the mean speed can be adjusted by hand through a considerable limit by lengthening or shortening the link H. For very close-speed regulation, as in driving alternators, etc., with heavy load changes, a compensating or "overtailing" device is fitted, either mechanical or electrical. This governor is suitable for operating needle nozzles, rectangular nozzles, and the gates of turbines, etc.

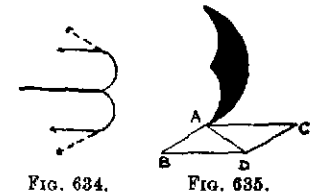


FIG. 634.

FIG. 635.

Mr Percy Pitman, Wilcott Road, Acton, London, W., is the maker.

It now remains for us to say a few words in reference to turbine bearings. Those having horizontal shafts have the usual type of bearing, which under ordinary circumstances would be of the ring-oiled type, and in very large machines of the oil-pressure type. Their construction does not much concern us here. Suitable stuffing boxes would also be provided, as we before mentioned. The usual design is that illustrated in the centrifugal pump in fig. 576, and described in the previous chapter. The smaller turbines which have a vertical shaft are so designed that this shaft runs on a footstep of lignum vitæ, or any other suitable design of footstep bearing, of which many ingenious designs, to avoid the friction of heavy machines, have been produced, notably a device whereby water is forced in under pressure at the base of the footstep, bearing the whole weight of the machine upon it, a constant flow being kept up past the shaft to keep the packing (of rubber rings) cool. A very good practical design of footstep is given in fig. 638. It is provided with a two-part housing B, tightened by bolts and the grubbing screw C.

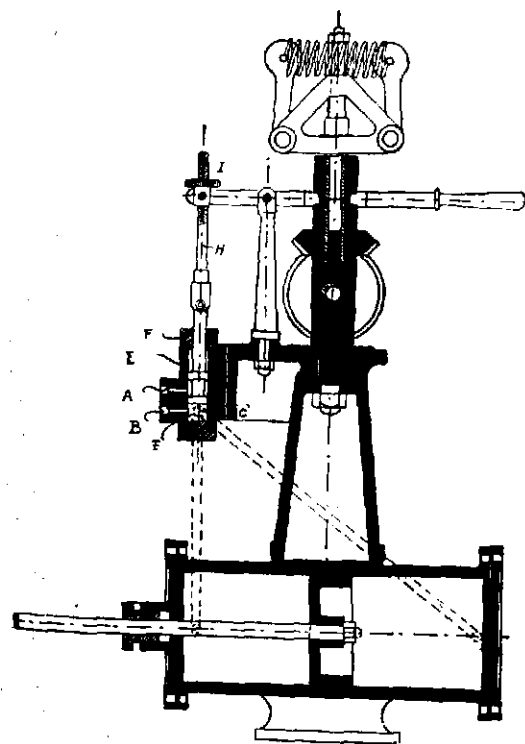


FIG. 636.

shaft, upon which in turn the turbine is hung. C is a bronze block (fig. 639) in which are grooves. Into these grooves, by means of the pipe F, oil is forced under pressure. The block B is then borne upon this by the oil film, and so reduces friction to a minimum. The bearing would rest on beams in the turbine-house. The following are the results of some turbine tests made in Scotland. They were selected as specially good examples, and will prove, I hope, of interest to the reader.

Records of Turbine Governor Tests at Kinlochleven and Blair Castle.

Kinlochleven.—The first set of readings are taken from a 500 H.P. pelton wheel working at Kinlochleven, which was driving generators supplying

FF are lignum vitæ bearing blocks attached to it, while E is a cast-steel bush, and D a stationary cast-iron journal attached to the foot-casting H. Vaseline is forced through G via J, L, and K to the shaft by means of spiral grooves in the blocks F. The usual practice of vertical turbines is to provide a lantern bearing, that is to say, a solid shaft is fixed in the base of the tail race, while at its top is a block on which runs a hollow

shaft, upon which in turn the turbine is hung. C is a bronze block (fig. 639) in which are grooves. Into these grooves, by means of the pipe F, oil is forced under pressure. The block B is then borne upon this by the oil film, and so reduces friction to a minimum. The bearing would rest on beams in the turbine-house. The following are the results of some turbine tests made in Scotland. They were selected as specially good examples, and will prove, I hope, of interest to the reader.

current to electric cranes and stone crushers. The power varied in a most extraordinary way; at times 200 or 300 H.P. was thrown off and put on again twice or thrice in a minute. The governor originally installed with the pelton, being entirely unable to control the speed, was disconnected and out of use, the speed being controlled by hand; this necessitated keeping two men

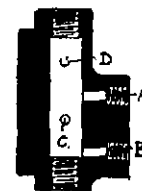


FIG. 637.

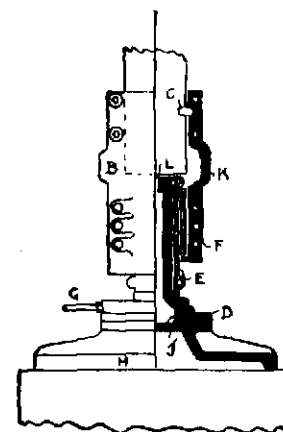


FIG. 638.

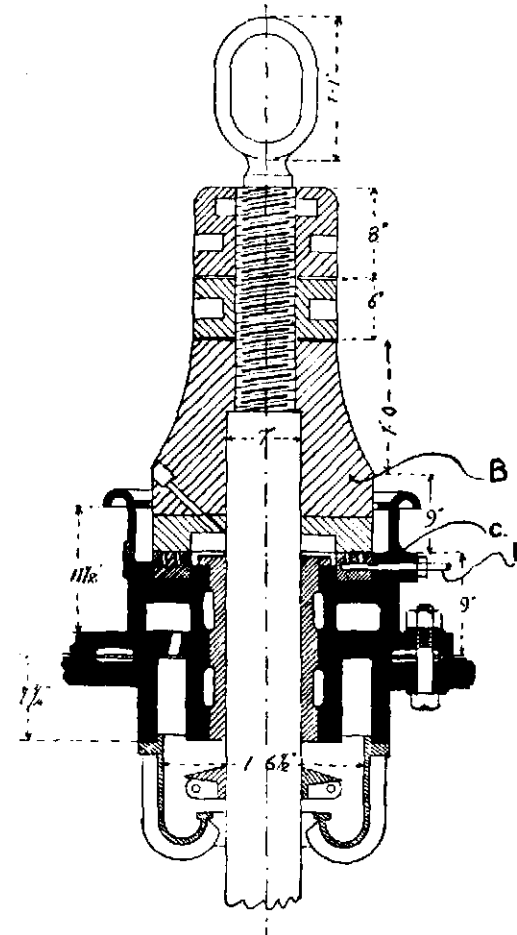


FIG. 639.

in attendance night and day. We* were then asked to take the matter up. The results given below show that the normal variation in speed was not more than 5%, although no fly-wheel was provided, nor was it possible to fix one without incurring great expense. The governing was effected by deflecting the jet, and there was therefore no question of shock in the pipe line. The governor was fixed without interfering with the works, which continued night and day, except for half an hour at midday. Directly the governor

* Messrs Gilbert Gilkes & Co., Kendal.

began to work it was recognised by the men working the cranes and other plant some miles away that the running of the expensive electric machinery was very much more steady and constant than had previously been the case,

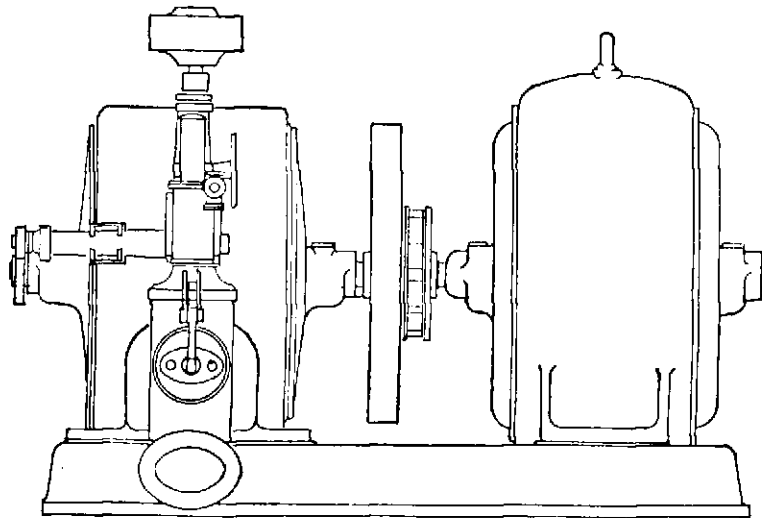


FIG. 640.

and the users of the governor (Sir John Jackson, Ltd.) expressed themselves well pleased with the results attained.

Although this governor was made to control the very heavy variations of load above referred to, it is at times called upon to regulate when running direct upon the lamps in the workmen's cottages, the turbine working at say 15 or 20 H.P.

Variation in Load.	Variation in Speed.	Load Variation.	Speed Variation.
Amperes.	Revs. per minute.	Per cent.	Per cent.
320—150	246—253	53	4.9
150—260	250—242	78	3.2
150—180	250—248	20	.8
180—300	250—240	67	4.0
130—220	252—242	70	4.0
200—90	248—260	55	4.8
90—150	253—243	67	2.0
300—60	247—260	80	5.2

Blair Castle.—Under the superintendence of Mr W. H. Massey, M.Inst.C.E., three 60 H.P. pelton wheels have been installed for the purpose of generating current for lighting Blair Castle, the historical residence of His Grace the Duke of Atholl.

The pelton wheels work under a head of about 400 feet, the pipe line

being over $1\frac{1}{2}$ miles in length. As economy of water is of importance, the governing in this case is effected by means of needle regulation in the nozzle. The figures given below were recorded during the normal house-lighting, and it will be seen that the variation in speed is under all ordinary circumstances almost negligible, very considerable variations in load only causing a momentary fluctuation of $2\frac{1}{2}\%$. These governors are provided with an isochronous arrangement, by means of which the normal speed of the turbine remains constant when working at either full, half, or very light loads. Each turbine is provided with a fly-wheel, and there is an automatic relief valve for preventing shock on the pipe line.

Plant running with a steady load of			A further load as below was thrown on or off as indicated.		Making a total load after change was made of	The speed rose or fell to	Being a momentary rise or fall of	And settled back to steady normal speed of 573 in
Amps.	Volts.	Revs.	Amperes.		Amperes.	Revs. per min.	Per cent.	Seconds.
			On.	Off.				
1	107	573	20	—	21	561	2.0	$1\frac{1}{2}$
21	107	573	Gradual of about 6 amps.	changes	—	570	.5	instantly
37	107	573	26	—	63	562	1.9	$1\frac{1}{2}$
63	107	573	8	—	71	570	.5	instantly
71	107	573	29	—	29	562	1.9	2
100	107	573	Gradual of 6 to 10 amps.	changes	—	570	.5	instantly
190	107	573	50	—	240	560	2.3	$2\frac{1}{2}$
240	107	573	Gradual of 6 to 10 amps.	changes	—	570	.5	instantly
300	107	573	"	"	—	576	.5	"
240	107	573	"	50	190	587	2.5	3
190	107	573	Gradual of 6 to 10 amps.	changes	—	576	.5	instantly
71	107	573	—	27	41	585	2.1	2
63	107	573	—	26	37	583	1.7	2
37	107	573	Gradual of about 6 amps.	changes	—	576	.5	instantly
21	107	573	—	20	1	585	2.1	4
1	107	573	steady	—	1	No change	—	—

CHAPTER XXVI.

MAIN DRAINAGE.

PERHAPS the most important duty of a municipal engineer is looking after sewerage and designing new works. Every town of any size and importance has some scheme for the removal of its domestic refuse and final treatment; in fact, the smaller hamlets and villages under rural district councils have also schemes now, because the Local Government Board are giving such facilities for public bodies to obtain loans for the proper carrying out of the work.

Perhaps the best way of describing to the young engineer the principles of main drainage will be to take him briefly through the several details connected with a sewerage scheme from beginning to end. We will suppose that a certain town is in need of a drainage system, and that an engineer has been called in to advise, report, design, estimate for, and finally carry out the works. The first consideration is the site of the town.

1. The inland town lying in a hollow of fairly flat surface, but the ground on all sides rising. There will generally be a river or stream of some sort flowing in the town, and along the banks of this will probably lie the main sewer or sewers if the river be large. The work in this case, which is perhaps the most frequently occurring, will not likely be a very hard matter. Many towns are similarly situated, usually at the mouth of a river, in which case the two main sewers run along each bank of the river to the disposal works or outfalls, where in the cases of London and Dublin the sewage has to be raised by pumping. In the former case there are two stations, one at Barking on the north side, and one at Crossness on the south side; while in Dublin, which has only lately had its sewerage completed, a syphon carries the sewage under the river Liffey at Butt Bridge, whence it flows onward by gravitation to the power station at Ringsend. The main drainage of London still stands paramount in examples of sewerage because of the magnitude of the work. In such cases as these the sewage usually gravitated towards the river and flowed out directly into it, with the general pollution of the banks; and the scheme of main drainage consisted in running main intercepting sewers along the banks to collect the various subsidiary ones, and transfer it far away down stream to the outfall works. Before Dublin received such attention the smell of the Liffey was a household word.

2. A flat surface in and all around the town. This will give rise to much ingenuity on the part of the engineer, because the sewers will be so flat that there will probably not be sufficient fall to take the sewage rapidly away, and it will probably be necessary to make artificial gradients by using small pumping units similar to Shone's ejectors, which we have described, or electrically-driven pumps. Southport is an example of a flat town.

3. An inclined surface on the side of a hill. There will probably be a watercourse at the base, and if land can be obtained on its banks for disposal works it will probably form an easy and economical means of sewerage; but when such land is unavailable (which is happily not very usual), pumping the sewage will have to be resorted to.

4. A flat surface on a hill top, perhaps the easiest drained of any, provided ample ventilation of the sewers be provided.

Then when the engineer has finally decided which heading the scheme is likely to fall under, he must procure accurate maps of the place. In England he will find the 5-ft. and 10-ft. Ordnance maps of great value, but if he can so find a map having contours, so much the better, and he can then decide upon the outfall works and the general run of his sewers. The next thing to do is to find out the amount of sewage he will have to deal with at the present and in the future. This is perhaps the most important item of the work, and demands careful searching of rainfall records. The presence of springs and subsoil water, the accurate determination of the present population, its recent annual increase, and hence probable future increase, which may so depend upon various circumstances of benefit or detriment to the town, and finally the average daily water supply per head of the population, all demand attention.

Of course, in the event of not being able to obtain any suitable maps, a survey of the whole town would have to be made. An important thing to consider, which might not at once occur to the engineer, is the liability of his district to subsidence. Coal-mining districts are always liable, and also salt mines, because large subsidences are continually occurring, which so seriously alter the levels of the district that the levels on Ordnance maps and the contours, when applied to such districts, are often found to be entirely unreliable, and then a survey is absolutely essential. In fact, Ordnance map levels are only to be relied upon for a first approximation, never for practical carrying out of work. In the open country, however, they are usually very accurate. I am not in any way reflecting upon the excellent work done by the Ordnance survey; on the other hand, it is a marvel of accuracy; at the same time, the latest information is naturally not always available.

By aid of the 6-in. contour maps the drainage areas may be easily determined provisionally. The next thing to do, with each separate area or drainage (a large scheme is usually divided into such areas), is to lightly colour them upon the 6-in. map in distinctive colours, and then take out the draught very carefully by any of the methods described in a previous chapter, or by that useful instrument, the planimeter. Of course it is entirely a matter for the engineer to decide the outfalls of the various areas, because a very important consideration is the main outfall, especially in inland towns, where the site of the purification works must be on a soil suited to the purpose, at least in many cases. We shall discuss this later on. For instance, there may be a very low-lying piece of land quite unsuited for an outfall. It may also be of no great magnitude. Obviously, then, if a suitable piece of land, which can be purchased at a moderate price, appeared at a somewhat higher level, it would be prudent and economical to raise the low-level sewage by pumping or other means. When the proposed lines of sewers have been finally decided upon, we must find out what the character of the subsoil is. If, for instance, we find a sewer is going to pass over a piece of very waterlogged and boggy land, its direction may have to be altered to save expense of driving the sewer in tunnel under compressed air. The next thing to do is to

find where we are going to place the storm-water overflows. In intercepting sewers laid on the banks of rivers or near to them, the original outlets form natural positions in themselves; in other cases small streams will serve the same purpose. On the other hand, special culverts may have to be built. But they must be above the storm or high-tide level of the river, so as to prevent floods rising up the sewers and flooding basements of houses, etc., a very frequent occurrence. For the same reason, the level of the outfall works must be above the storm-water level. The writer knows frequent instances of this in Dublin, many of the large business houses in the city having suffered from such flooding before the main drainage was completed, special appliances having to be resorted to to avoid it, and such appliances are always unsatisfactory at the best of times and are well let alone. In fact, anything complicated in a drainage or sewerage system must be strenuously avoided. This will primarily determine the level of the main sewers, but these sewers must also be capable of taking all water from basements, etc., at least 10 ft. below the road surface, and it is these two conflicting elements which present such difficulties to the drainage expert, especially if he is striving (and rightly so) for a gravitation system, bearing in mind that the sewers must be laid to have a self-cleansing velocity. Assuming the areas to be correctly arrived at, the next thing to do is to see if we cannot divert some of the surface water into the watercourses or other place of disposal, to avoid filling up the sewers. Having done so, we find the total area of surface water which will have to go into the sewers, and this includes the roofs of all the houses on the area. Now, taking the past records of increase of population in any district over thirty years, we can generally conclude, unless we have any special reasons to act to the contrary, that the next thirty years will increase the same way, thirty years being the usual time to consider, as the L.G.B. fix the repayment of loans over thirty years.

Now, from statistics, it has been found that there is, on an average, 1 house to every 5 people. Therefore the population is a direct measure of the number of houses in the particular district approximately. The area of these houses will, of course, vary within wide limits; 160-450 sq. yds. may be taken on an average, but 340 would perhaps be nearly a general approximation, seeing that the average number of people to the acre in ordinary suburban districts = 70. Now it is very important to find out how much sewage we are going to deal with. Domestic supply has been shown to be very nearly equal to the water supply of the town, that is, 12-25 gallons per head per day. But this is not all. Subsoil water, surface water, and manufacturers' waste play a very important part in swelling up the volume, and the definition of the exact average quantity passing is a very hard question. Subsoil water must be kept out at all costs by properly designed sewers. Manufacturers' waste can usually be gauged at each works. Then there is the rainfall. We want to consult available statistics or make local gaugings and find the inches of rainfall over a considerable time. Symons' *British Rainfall* may be consulted with advantage. Then if we have the rainfall and the built-on area (we are assuming all other lands drain naturally into watercourses) we can calculate the average daily discharge, which, of course, will fluctuate enormously. For instance, the average rainfall may be 26 in. per year, but a very heavy storm may supply a downpour lasting half an hour at the rate of 72 in. per day. The only way to get satisfaction is to get records (and good ones, too) and find out the average hourly rainfall. Then, again, in some places the surface water may descend very rapidly into the sewers. On flat

macadam roads part of it may get lost or evaporate. However, on a paved street or an asphalt pavement we may expect nearly the whole of the rainfall to enter the sewers, especially in the case of a very sudden downpour. Let us investigate a case. Say we have a built-on area of 80 acres = $\frac{1}{3}$ of a mile; we are allowing for a crowded population of 10,000 people on the area and a water supply of 20 gallons per head per day.

$$\therefore \frac{20 \times 10000 \times 365}{6 \cdot 25} = 12,000,000 \text{ cub. ft. per annum.}$$

We now have to add the rainfall. We are going to assume a fall of 24 in. per annum over the whole surface to be removed = $2 \times 4840 \times 9 \times 80$ = about 6½ million cub. ft. of rainfall to be removed per annum = 18½ million cub. ft. of sewage to be removed in a year. Now we know there are 525,600 minutes in a year, $\therefore \frac{18500000}{525600}$ = about 35 cub. ft. per minute as the average dry-

weather flow. We can only give this as an example: so many different circumstances crop up in special cases that no hard and fast rules can be laid down. Another thing which will help to determine the size of any particular sewer is the composition or strength of the sewage at the outfall. It must be analysed for albuminoid ammonia or oxygen absorbed, which on an average will be 1½ grains per gallon. Now the Local Government rules in reference to sewage going direct into streams demands that it shall be diluted before entering the said river down to .04 grains of albuminoid ammonia per gallon. Therefore we should have to dilute out 35 cub. ft. to a flow of $35 \times 37 \cdot 5 = 1312 \cdot 5$ cub. ft. per minute before we could allow it to go into a river by means of a storm overflow. But the Local Government Board requirements are that 6 times the average dry-weather flow. $\therefore$ the 35 cub. ft. $\times 6 = 210$ cub. ft. of sewage to be taken and treated at the disposal works. Any quantity over and above this may go direct into the streams by the storm overflows. Now our 210 cub. ft. per minute = 3.5 cub. ft. per second. Now we know from Hydraulics that a circular sewer has a velocity in ft. per second = $V = 13 \cdot 725 \sqrt{ds}$, d being the diameter of the pipe in ins. and s = the fall.

Assume the sewer to have a fall of 1 in 240 = $\frac{1}{240}$.

Then a 18-in. sewer would deliver 220 cub. ft. per minute when running half full. Therefore that would theoretically be the size of sewer we should adopt for the population of 10,000 people on $\frac{1}{3}$ of a sq. mile. Say we had to deal with a similar case of an area of 1 sq. mile, $210 \times 8 = 1680$ cub. ft. per minute. We should probably use an egg-shape sewer of the form shown in fig. 519. Then if d = the diameter of the larger circle, the discharge of the sewer in cub. ft. per second

$$= Q = .08 \sqrt{d^5 s} \quad \dots \quad (394)$$

We shall find that a similar sewer, if the fall was 1-100, would have to be about 3 ft. in diameter to carry off the quantity running $\frac{3}{4}$ full. The sill of the storm-water overflow would in the case of the 18-in. sewer be at 9 in. above the invert, and for the 3 ft. egg-shape sewer 4 ft. to 6 ft. above the invert.

We shall now give an example of a section for a sewer, and also a plan showing the various details thereon. The plan and section are shown in fig. 641, the manholes being shown. It is usual to draw such sections when the plans have been prepared in the usual manner as before described under Levelling. When this is done the engineer can see what is the best gradient

sewers. For the branch sewers 6-in. and 9-in. pipes will suffice. Gradients of 1-36 to 1-112 will be in general satisfactory. Above this there will be a liability of the solids being left high and dry in the pipe, while the liquid will pass away, and a flushing-tank at the top end will probably be required. Again, where the grade is flat, similar artificial means will have to be provided.

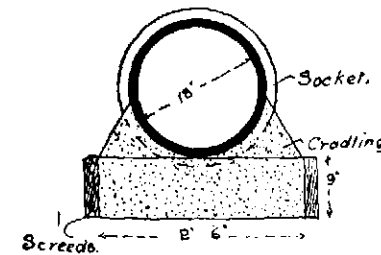


FIG. 642.

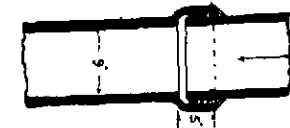


FIG. 642A.

1-240 is about the limit of efficiency in pipe sewers of the circular form. All junctions in pipes must be splay, viz. inclined about 60° towards the flow, so as not to impede it. When a pipe sewer joins a built one the junction must be very carefully made. Special junction blocks are to be advised. It is interesting to know that

a 10 ft. diameter sewer with a fall of 2 ft. per mile	
" 5 " " " "	4 "
" 2 " " " "	10 "
" 1 " " " "	20 "

all have the same velocity of flow; but to produce this flow the volume flowing in the 10-ft. sewer must be 4 times that in the 5-ft. sewer, 25 times that in the 2-ft. sewer, and 100 times that in the 1-ft. sewer. No pipe sewer must run more than half full under ordinary circumstances. Now a form of pipe sewer is shown in fig. 643.

It must lie on a concrete bed, laid in screeds at the bottom of the trench and finished hard. The screeds may with advantage be left in. The joints of the pipe should be made as follows. When the bed has been truly set to the proper gradient, by means of boning rods, etc., described in a previous chapter, the pipes are all laid with sockets facing the flow. They are then set truly concentric with each other by means of tarred gaskin carefully packed into the sockets. Then a mixture of neat Medina cement is carefully packed into the joints likewise and trowelled off. It must only be mixed in quantities enough for 2-3 joints at a time, and no water added after mixing. The inside of each pipe is carefully scraped to remove any loose cement. When

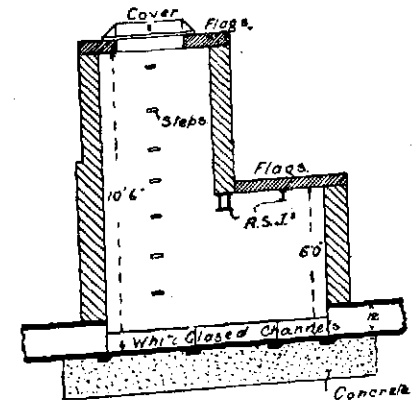


FIG. 643.

at which his sewer is to run, and the best position for his storm overflow. It may here be mentioned that most towns at the present day are seweraged on the combined system, viz. all the surface and other water as well as domestic sewage enters the sewers and goes away to the outfall or disposal works. Then there is the separate system, adopted in only a few cases, but advocated by many engineers. Here there must be two sets of sewers, one for the domestic sewage, which takes it away to the disposal works, and an entirely separate system for the storm waters.

That such a system does not work well is clearly evinced by the small amount of attention it has had. The author can find absolutely no recommendations for it; the expense of such a system is enormous; it is nearly always the case that the surface waters would be almost as foul as the sewage because of the filth necessarily abounding in the streets; especially in the first heavy rain after a long spell of dry weather must this be so, and what is to prevent ignorant people pouring their dirty water down the rainwater gullies instead of the special ones provided? One distinct advantage, of course, is the regular flow, which causes the actual treatment to be more satisfactory because no provision has to be made for storm water, and the stringent L.C.C. regulations of late have made engineers give more attention to the matter than previously; but as regards any resulting economy at the disposal works, from statistics it is rather doubtful if there is enough to warrant the extra outlay, except perhaps where there is much pumping necessary. On the other hand, in small towns the sewers would be very large compared with the ordinary flow; and unless the gradients were stiff they would be unable to maintain a self-cleansing velocity. When it is adopted, special means of occasional flushing must be provided or else the sewers get very foul, and, as before stated, the road washings are so likely to be foul that they must pass through settling tanks before passing into any watercourse. We now presume that the reader who we are taking through the details of a main drainage scheme has (1) mapped out his drainage areas and found their acreage

(2) That he has marked all the main sewers thereon.

(3) And taken careful levels over the whole system, and plotted sections of his sewers. It is also a great advantage to mark the heights above datum at various known points on the plan.

(4) That he has also plotted his branch sewers and defined their size, position, and available fall.

(5) That he has separately defined his unbuilt and "built upon" areas, and made due provision for the future.

(6) That he has marked the places for storm overflows.

(7) That a suitable site has been chosen for outfall works.

(8) And the nearest approximation made to the amount of sewage he has to deal with.

He will then be able to form an approximate estimate of the whole scheme, and will then probably have to submit preliminary drawings to the Local Government Board.

The next thing to do is to settle what form the sewers are to take. For the branches there is nothing to beat the fireclay or stoneware drain pipe up to 15 in. in diameter, and even 18 in. give satisfaction. It must be borne in mind that the sewers are to have a self-cleansing velocity, so as not to get sluggish. The daily flow, under ordinary circumstances, will of course vary greatly. It will usually be greatest (in dry weather) between 11 a.m. and 1 p.m., during which time $\frac{1}{4}$ to $\frac{1}{2}$ of the total daily flow will pass down the

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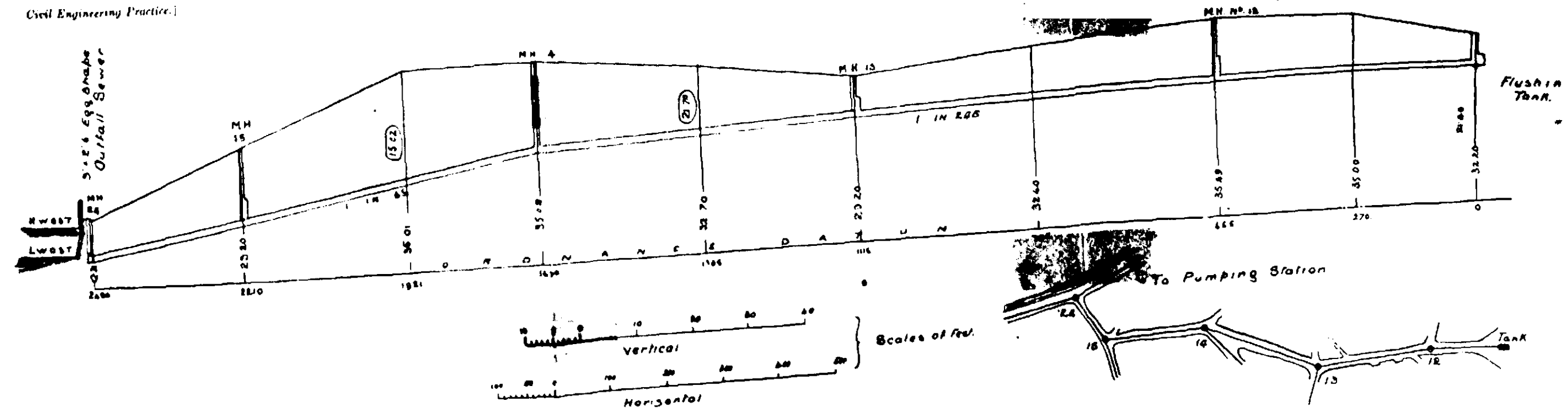


FIG. 641.

N.B. The figures would be written in red on the section. They denote the height above datum of the invert of the sewer.

the now. They are then set out concentric with each other by means of tarral gaskin carefully packed into the sockets. Then a mixture of neat Medina cement is carefully packed into the joints likewise and trowelled off. It must only be mixed in quantity enough for 2-3 joints at a time, and no water added after mixing. The inside of each pipe is carefully scraped to remove any loose cement. When

[To face p. 454.]

a length of pipe has been laid it must be tested as follows: Obtain a screw or bag plug and insert at the lower end. At the upper end place a bend in temporarily with cement, and socket turned upwards, then fill up the pipes till they will hold no more water, and wait till all the air has escaped, when the water should stand quite steady. If not so, a leakage is occurring, and it must be found and rectified, and the process repeated. It is most essential that the engineer sees all this done himself. At each change of direction a manhole must be provided. These may be circular, rectangular, or conical in form, as circumstances will demand. A form is shown in fig. 643. A cast-iron ventilating cover must always be provided, and steps to gain access to the sewer. To be of any use, a manhole must be at least 5 ft. long, but it may be 2 ft. 6 in. to 3 ft. wide. All pipes must be extra salt-glazed. The providing of lamp holes, although considered by some engineers to be of advantage, seems to the writer to be rather a needless expense. When the manholes have been built and the sewer finished it may be then filled in, but before doing so the

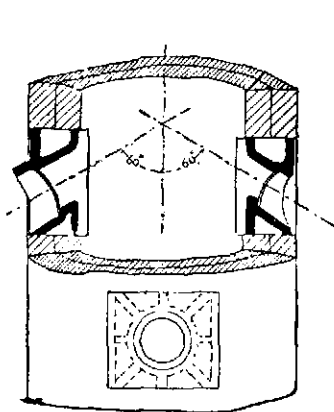


FIG. 644.

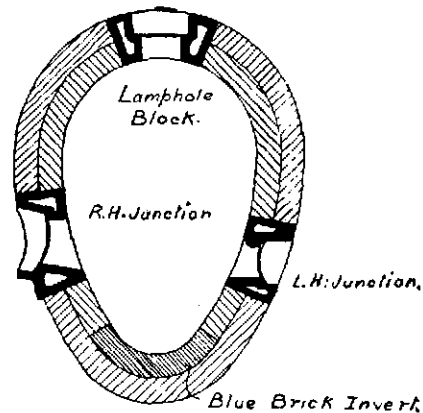


FIG. 645.

pipes must be cradled or banked up to at least half their height with concrete, which must be very carefully packed *underneath* the pipes, to take the weight off the sockets and prevent rupture under the superincumbent load. The filling in must be done in layers, carefully rammed and watered.

Numerous spare junctions should be provided in a new sewer to obviate breaking into it again. They must be stopped with a piece of slate and mortar, and directly above them a mark should be made on the kerbstone or other place, and duly referred to in the engineer's office, so as to obviate hunting for it in the future. It is most essential to treat all junctions in this way. We stated, pipes up to 18 in. can be conveniently used; above this, built culverts are used to more advantage and economy. We may have various forms, but the circular sewer and the egg-shape sewer seem to be the usual. They may be of brickwork in cement, concrete, reinforced concrete, or cast-iron segments, similar to tube railways, flushed up inside with cement concrete. This practice is only used in water-logged ground. For ordinary purposes the egg-shaped sewer stands paramount. The object of the design is that this form adapts itself to very flat grades by supplying an artificial head to give velocity to the flow. Again, as the floods rise, so the level of water in the

sewer rises slower in proportion. Altogether it is a very favourite form, and is shown in two views in figs. 644 and 645, with special lamp hole and junction blocks. The construction requires some care, especially the invert; and it is quite the best practice in brick sewers of this kind to lay special invert blocks set on a concrete bed, truly to the proper gradient, the sewer springing therefrom. The brickwork must be laid in cement. The tendency is now to make egg-shape sewers with a somewhat smaller radius to the invert than previously, *vide* fig. 645A.

Then we have the circular sewers for larger sizes up to 12 ft. in diameter. They may be of brick or concrete, with or without reinforcement, and are built the same way as any circular culvert. In either case the invert is best made of very hard Stafford blue bricks, as there will always of necessity be a certain amount of erosion continually taking place, due to the flow of sewage. They should be carried up to $\frac{1}{2}$ the height, and the same remarks apply to the egg-shape sewers, whether or not an invert block is provided. All the joints must be very carefully struck and examined by the engineer himself for good workmanship. Cement mortar 2-1 should always be used. A penknife poked into such joints will usually reveal faulty workmanship. In no case can brick sewers be entrusted to ordinary workmen. Bricklayers accustomed to this special class of work must be employed. It is usual to provide a concrete bed, and sometimes the concrete is carried up (in loose ground) to half the height of the sewer. This, then, if carefully laid to a proper shape by means of moulds, provides a good bed for the brickwork, but in very firm ground it is not necessary, and there the excavation of the trench requires some care. Special templates must be cut, and the excavation very carefully cut to this shape, any *superfluous* stuff removed being replaced by concrete *only*. A special form for the concrete work is shown in fig. 646, and one for the brickwork in fig. 647, the line of each course being carefully marked thereon by means of small nails. The engineer must check the invert levels at intervals. Large sewers very often take various forms. The main outfall in London on the Barking side consists in parts of three parallel tunnels laid on an embankment over the Erith marshes. Special forms of large sewer are shown in figs. 648, 649, 650.

We have already advised the use of special junction blocks of bell-mouth shape for bringing pipe sewers into the mains. But when two main sewers of large dimensions join, the work demands very special care, because of the large flows, and naturally flat gradients; a badly formed junction will only impede the flow, and cause an ever-increasing deposit of silt, necessitating periodical removal by hand. A good sewer, properly laid, should never need any such attention.

The ventilation of sewers now demands attention. There are a few engineers who are opposed to its adoption, but the author is strongly in favour of thorough ventilation for all sewers. Those who differ contend that a good

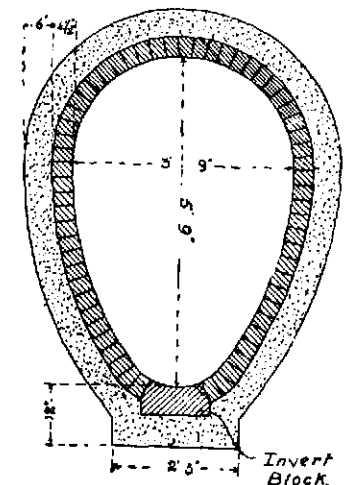


FIG. 645A.—Egg-shape Sewer.

however, have to be adopted in narrow courtyards where surface gratings would be a nuisance; their effect may be assisted by a Boyle's "air pump extractor," but fancy features of any sewerage system are always to be avoided. It has been stated on good authority that the air in a sewer purifies itself by undergoing a sort of bacterial treatment. Whether such is the case or not is doubtful, but the writer would be inclined to concur with the views. Where

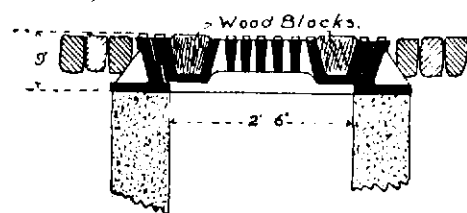


FIG. 651A.

bad smells and gas emanate from a sewer, especially those manholes at the top of a long and steep sewer, it may be necessary to install a charcoal deodorising ventilator. Latham's is perhaps the best design of the kind and is shown by fig. 652. The charcoal must be kept dry, and when it is so it will

decompose H_2S and ammonia by a process of oxidation, and make the foulest air harmless. Artificial ventilation of sewers cannot be advised, but a rather good way of assisting sewer ventilation when Shone's ejectors are installed (see figs. 588-591) is to turn the exhaust air into a sort of ejector,* which at each working draws with it a large quantity of sewer air without extra expense. It also has the advantage of dealing with a quantity of bad air proportional to the flow of sewage. There are also various chemical systems of foul air purification. The process can hardly be called one of actual purification, however; it is merely deodorising or rendering the foul air innocuous. In the writer's opinion they are not to be recommended, being only additional expense in outlay and maintenance, and a nuisance to look after. For the sake, however, of making our paragraph complete, we will briefly describe two forms of apparatus. First we have Reeves' system, which consists of two earthenware pots placed in the manhole chamber. One of these is filled with strong H_2SO_4 , while the other contains manganate of soda. Below is placed another pot. Water is allowed to drip slowly on to the manganate of soda, the solution falling into the lower jar. Into this also at a suitable predetermined rate trickles the H_2SO_4 . A chemical action takes place, in which large quantities of permanganic acid are evolved, the gas itself being a very powerful deodorant. A very fine water spray is introduced which causes a sort of mist. This, falling by its specific gravity, meets the upward flow of gas and so purifies it, while the solution which falls into the sewage assists

* Better called an exhaustor.

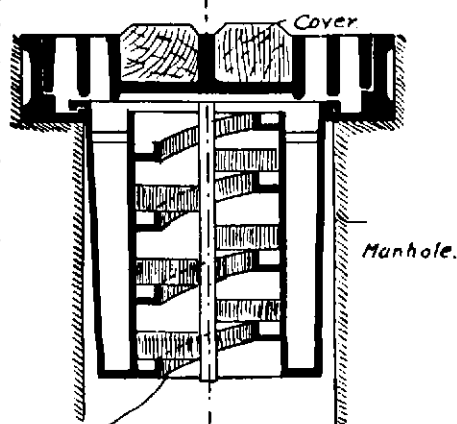


FIG. 652.

Spiral Trays Filled with Charcoal.

in oxidising the same. The apparatus costs about £10, 10s., and would cost from 45s. to 50s. per annum for upkeep.

A simpler system is Stone's. A water jet in a sort of conical arrangement acts as an ejector and draws the air out of the sewer, washes and purifies it, and finally discharges into the atmosphere. The water runs to waste.

Most sewers, especially those with very flat and very steep gradients, must be supplied with suitable apparatus for periodical flushing and removing obstructions. The simplest way to perform the duty is by placing at intervals, preferably with manholes, hinged gates or doors of cast iron or suitable penstocks. By closing these doors or penstocks the sewage behind is dammed up, and when suddenly released rushes forward with considerable velocity, due to the extra head, and will effectively remove obstructions for some distance down the sewer till hydraulic laws have brought the flow once more to its natural velocity. Such an apparatus is shown in fig. 653, applied to a 12-in.

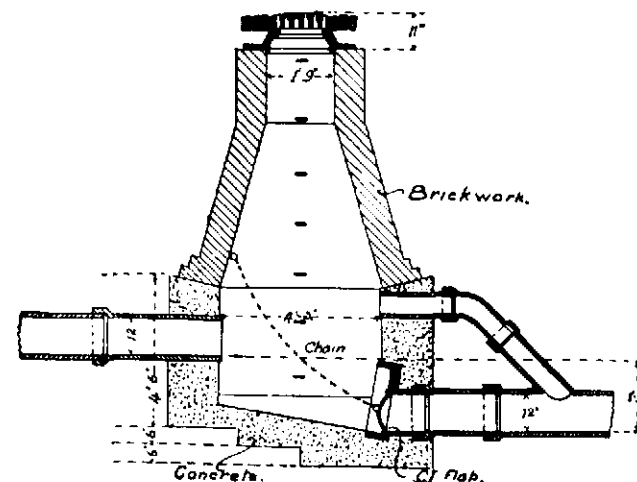


FIG. 653.

sewer. The gate or flap is of cast iron and hangs vertically, and is attached by a chain to the walls of the manhole; an overflow is also provided. When the sewage has reached the highest level it can, a man descends into the manhole and pulls up the flap by the chain, and fastens it securely. Under great heads this becomes an unsatisfactory way of doing things, owing to the difficulty of raising the gate and liability to fracture.

We then resort to the sluice gate or penstock, shown in figs. 654 and 655, which is raised by a screw and worm-wheel motion. The smaller forms simply provide a handle for pulling up the sluice in a vertical plane. They are always liable to stick if not frequently used. The joints should be faced and well covered with tallow. Some manholes simply have grooves into which boards are inserted to dam back the sewage. Their removal, however, is difficult and unsatisfactory.

A more modern way of flushing sewers is by means of the flushing tank and syphon. Pure water or sewage can be used in the apparatus, which is shown in two views in figs. 656 and 657. It essentially consists of a large brick or concrete chamber into which water or sewage flows. In this chamber

is placed a cast-iron syphon and discharge pipe. As the water rises, so the air is discharged from the bell. A dribbling then takes place, which finally exhausts all the air, when the pressure of the atmosphere causes the contents

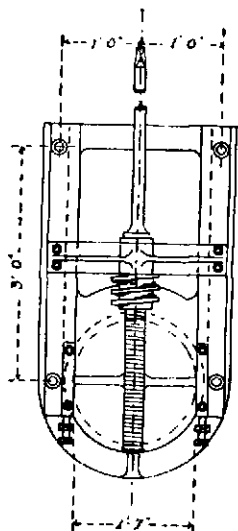


Fig. 654.

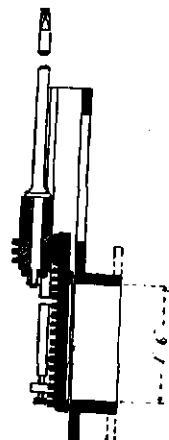


Fig. 655.

of the whole tanks to be suddenly discharged at considerable velocity. The action is similar to the ordinary w.c. cistern, only the pull of the handle actuates the cistern, while the flushing tank syphon is automatic. By lifting

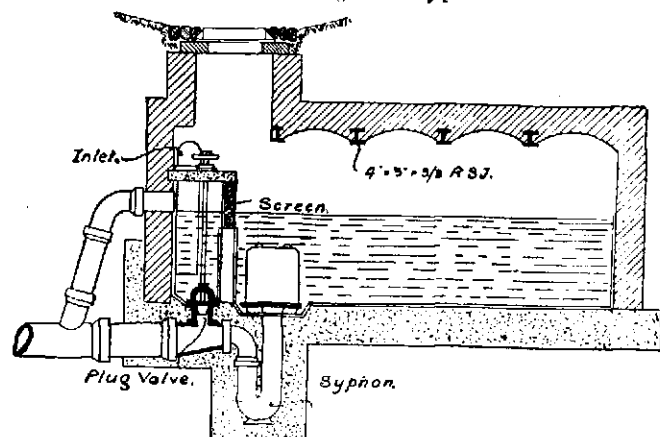


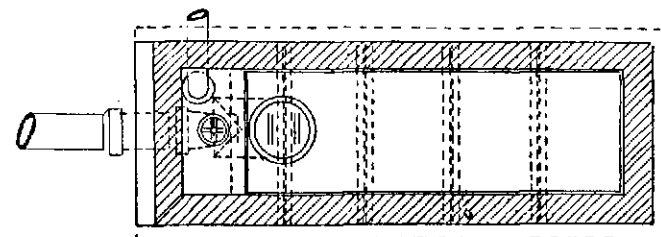
Fig. 656.

-SECTION.-

the bell, however, it can be made to act at any time. The tank shown was designed for taking sewage. A screen is provided so as to prevent foreign bodies from choking the syphon. An overflow must always be provided, too, in case of failure of action. The refuse which collects at the bottom of the

side chamber may be let off periodically by the plug valve. It is preferably done before a flush is sent down. To make a syphon discharge effective the length of the shorter leg of the syphon must be equal in length to the depth of water above the bell at the level it is required to start discharging. For very large sewers, however, flushing tanks, in order to have any practical effect, would have to be so large that they usually give way to the use of flushing gates. It may here be remarked, however, that, given favourable circumstances, an engineer should so be able to design his sewers as to give efficient self-cleansing velocities.

Tide-locked sewers, however, have a strong tendency to silt up, because they are usually of the same sectional area as if they were not tide-locked, whereas



P L A N.

Fig. 657.

if the engineer decided upon reducing the sectional area in the tide-locked portions about one-third, there would be little or no tendency to silt up, while if he has a sewer silted up and rebuilds it of a sectional area equal to that which it had when silted up, it will probably remedy the evil. Where the sewage discharges into the sea, it may be deemed prudent only to let it off on the ebb tide, for which purpose specially large sewers have to be provided to store up the required quantity. They are called tank sewers, and a form of such is shown by fig. 658. Before adopting a tidal outlet, however, it is most essential to find out the exact direction of all local currents, so as to avoid dangerous deposits. For the purpose of allowing the sewage to flow out on the ebb tide only, hinged flap valves have to be provided. They are made in all sizes and shapes, but they must have two planed faces and be hung on bronze bolts with bronze nuts. The discharging pipe must be ventilated. We before pointed out the undesirability of a sewer having too great a fall. Where possible the gradients should be reduced by allowing it to fall a few feet vertically at various places. Such may be effected by a special cast-iron stand pipe, but the invert of the sewer must be specially hard to prevent erosion due to the continual fall of water, for which purpose it is usually made of Stafford blue bricks or cast iron. A form of such a manhole is given in fig. 659, but an equally effective system is shown in fig. 660. We now have to consider the practical construction of storm overflows. It was before stated

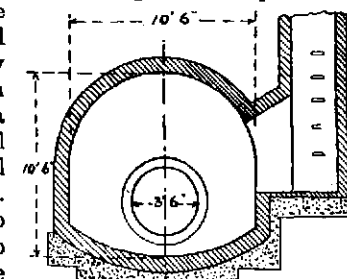


Fig. 658.

that when, under ordinary circumstances, the level of sewage in the sewer rises to more than six times the dry-weather flow, the superabundant volumes may go unpurified into a ditch or stream, if it is not likely to cause any nuisance therein. A good form of such an overflow is illustrated in figs. 661 and 662. There is another form wherein the action of a float upon a small sluice closes off a portion of the main sewer according to the rise of water therein. The system is that advised by Mr Wood in his *Sanitary Engineering*, and is a

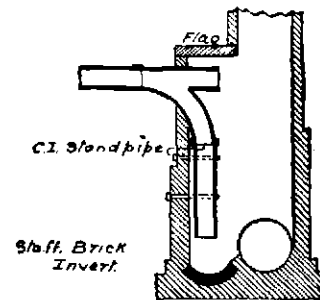


FIG. 659.

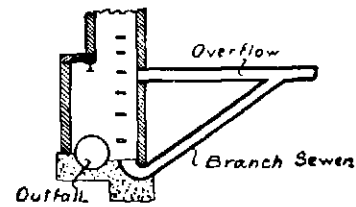


FIG. 660.

good practical design as long as the floats remain in proper working order, to ensure which it should be subject to periodic inspection. A simpler form of weir is shown in fig. 663, and yet another form is the parabolic leap weir. This weir is of scientific design, acting as it does upon the velocity of the water due to the head. Its principal use is in waterworks schemes, where a sudden flood is liable to bring down very discoloured water, and is by this

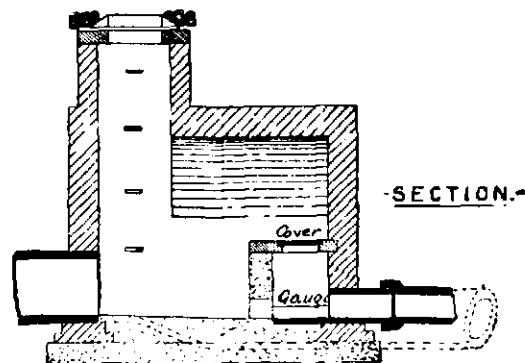


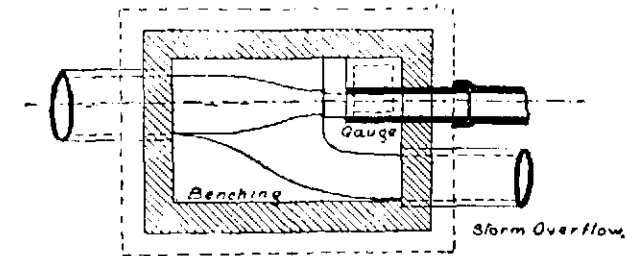
FIG. 661.

means rejected from the reservoir supply and goes away into the by-wash. It must have a good fall on the main sewer, however, to become effective. The principle of the design is as follows, referring to fig. 664. A is the main sewer going to the outfall. B is the storm overflow. Now the engineer has to determine what depth of water there will be at B when six times the dry-weather flow is coming down. The effective head, however, can only be considered at $\frac{1}{3}$ of this value. Then the levels of the two sewers will probably fix the value of the distance x . Say it is 1 foot, and say the effective head = h . We only have to determine the value of y , so that all water

above six volumes will flow over the lip B, while the ordinary flow goes away to the outfall. Its value

$$= y = \sqrt{1.48 h} \quad (395)$$

Now it must not be supposed that sewerage a town is merely laying sewers at proper gradients under streets or other places, because many difficulties will present themselves to the engineer in a big scheme. First of all, we may have to cross a river. If the stream be small and the levels satisfactory, it may be carried over in a brick culvert, in a weir, or upon arches,



-PLAN-

FIG. 662.

or a pipe may be laid on a special bridge of reinforced concrete or other material, which will only be a modification of an ordinary road bridge. The place also forms a very convenient spot for a storm overflow.

The details of the design and construction of some special cases of taking pipes across rivers appears in the chapter dealing with Steel Construction. A very simple case, however, suggested to me by Mr Arthur Bowes, M.I.C.E.,

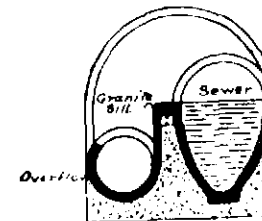


FIG. 663.

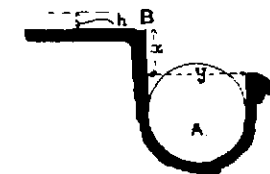


FIG. 664.

Earlstown, Lanes, is given in figs. 665 and 666. The drawings are of practical value.

There may be cases, however, where a sewer will have to be taken under a river by means of an inverted syphon. An example of a large piece of work of a similar nature will be seen in the syphon at Butt Bridge, Dublin, in the new main drainage (1907) scheme, where the whole of the sewage from the north side of the city passes to the south, and on to the Ringsend pumping station. It was made under much difficulty, and the work reflects credit on Irish engineers. The elements of such a syphon are shown in fig. 667. The principle is of water finding its own level. The pipes would be of cast iron,

flanged, faced, and bolted. Rubber ring packing could be used. The essentials of design are as follows:—

1. A self-cleansing velocity must be induced by providing a total fall between the two manholes of at least 1-80.

2. To effect this, it is better to lay two or more smaller pipes than one large one, coming into action automatically. At the lowest level, as shown, a cleaning valve and shut-off valve must be provided; also, if possible, a means of dealing with the sewage in case of breakdown. It may be necessary to provide a tank to collect the sludge when cleansing, and a small electrically driven centrifugal or diaphragm pump for its removal, or if compressed air is handy, a Shone's ejector. It may be flushed in the usual manner. Another way of crossing a river, if practicable, is that shown by fig. 668. The necessarily difficult and expensive part of the work is obviously the cofferdam.

Then again, in big cities, it may be necessary to tunnel and lay the sewer therein.

The procedure is generally adopted below 20 ft. in depth. We have already described the setting out of tunnels in a previous chapter. We shall now say a few words on practical construction.

Having surveyed the work, the next thing to do is to sink the shafts at distances considered most convenient. Equal distances are advisable. They must be sunk at changes of direction. In firm ground the excavation is carried down to a safe limit, and a circular ring or curb of timber placed therein, and the brickwork built upon it to the surface.

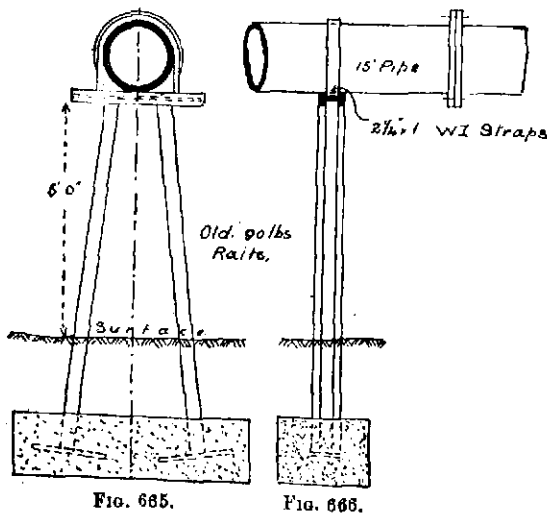


FIG. 665.

FIG. 666.

Inside this curb the excavation proceeds, the previous curb and brickwork being supported on props. The procedure is shown by fig. 669. In soft ground, however, a cast-iron or steel cutting curb has to be provided. Wood curbs must be made of oak or elm.

A cast-iron curb is shown in fig. 670. In all such shafts it is very essential to carefully ram the earth in well behind the brickwork. As the excavation is proceeded with, the brickwork, freed of resistance except friction, will (or should) sink gradually. The engineer in charge must take great care that it sinks evenly, or else it will get out of the perpendicular and stick, which is sometimes fatal, although it can generally be moved by a water jet applied to the ground outside. When the lower parts are reached the excavating must be very carefully done, the curb supported, and the sides timbered. The heading must now be started at the bottom level. Rock tunnelling is naturally the safest material, but it may be very hard and slow to work; clay is not so good; but the worst are quicksand and mud.

The bottom of a shaft forms a sump, from which the water is pumped to the surface, either by steam-driven pulsometers or by electrically-driven

centrifugals, and to which it is directed by gravitation. Headings must be at least 3 ft. wide by 5 ft. high, and timbering will usually be necessary. We are only briefly describing tunnelling, as it is a work which demands special attention and study, and, above all, practical experience. With sewers, much of the timber is usually left in; it is an advantage, and the old timbers do not count for much. However, in large sewers, the brickwork may go on as the excavation proceeds. The timbering for a circular sewer in two stages

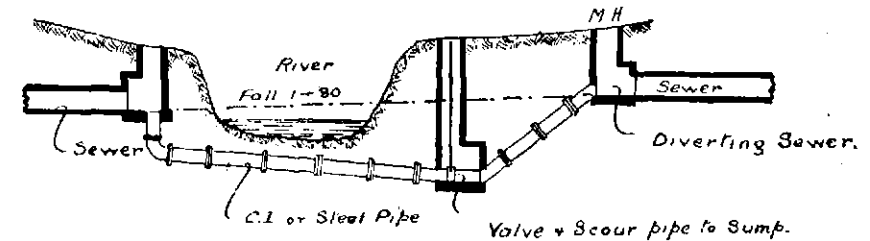


FIG. 667.

is shown by figs. 671 and 672. With pipe sewers, however, it is usual to make a tunnel from end to end, large enough for men to work it and timber it up, and lay the sewers therein. The method is shown by fig. 673.

Before leaving the subject of main drainage, we must consider a somewhat ingenious piece of mechanism for raising sewage without expense. We have already described pumping engines, but it remains to give a brief account here of Adams' patent sewage lift, which employs the head of a flow of sewage

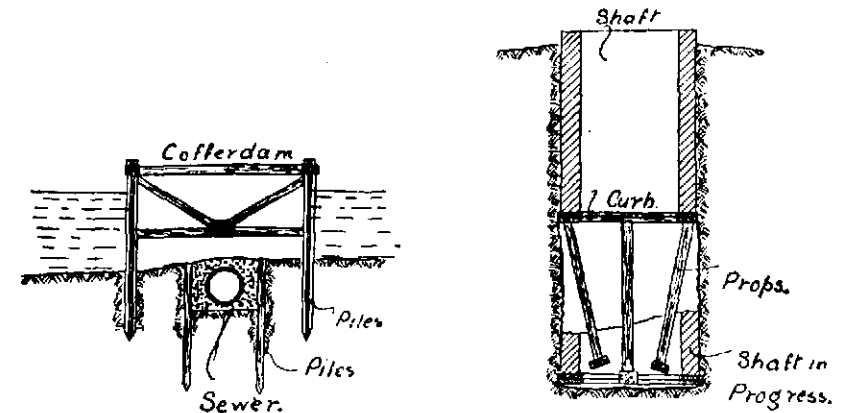


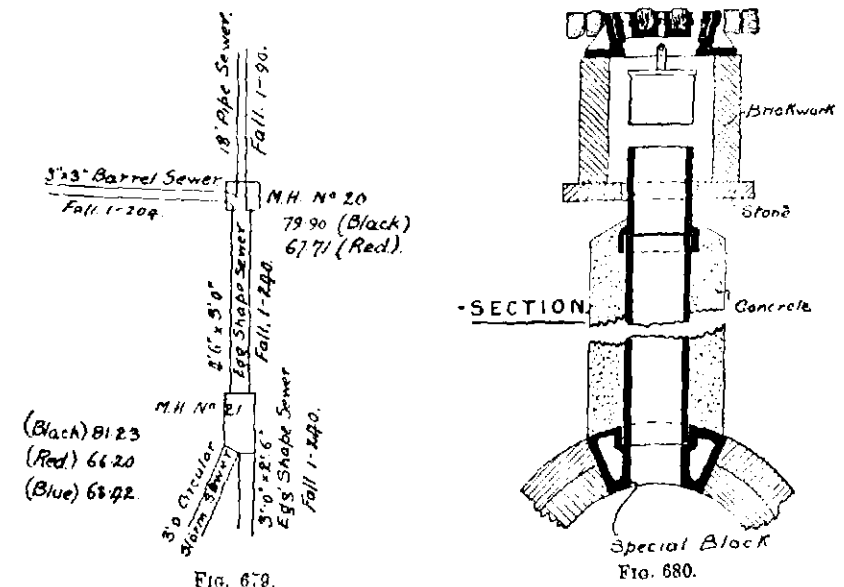
FIG. 668.

FIG. 669.

from a high-level district to raise the sewage (a smaller quantity usually) from a low-level district. The details of the apparatus are shown by figs. 674-5-6, while a block plan and section of the system are shown by figs. 677-678. First of all, we have sewage from a high level flowing down into a catch-pit, where the heavier solids and foreign bodies are caught. Thence it flows into a flushing chamber. When the water rises to a predetermined height the syphon discharges it rapidly into the air cylinder at a lower level. This causes a rapid displacement of air from the air cylinder at considerable pressure, according to the head of water behind it. This air finds its way by a steel

Fresh water may be applied to the same purpose, but naturally causes working charges.

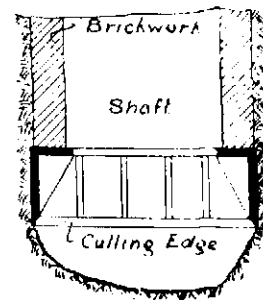
Of course, needless to say, the completion of a main drainage scheme for a town does not complete the work thereon by any means. We are going to discuss Sewage Disposal in the next chapter. But the maintenance of the scheme takes a considerable part of a municipal engineer's time to keep everything in order. In the first place, a good map showing the sewerage system *only* is indispensable. In large towns, of course, there is a special engineer whose duty it is to look after such maps and sewers only, but in the smaller towns it may be left in the hands of the chief assistant, who no doubt would take equal interest in the work. As before advised, Ordnance maps if possible should be used for the purpose, and all the information possible laid down thereon. All spare junctions must be very accurately marked on in reference



to existing buildings, etc., and also distinctive marks made at the actual place.

First of all, the main sewers must be shown in their proper direction by two parallel lines in red ink, the space in between them being coloured red. The manholes would be shown in the same way, according as their shape was circular or rectangular. On the sewer, in red letters also, should be written the size of it. In black should be written the inclination of the sewer, 1-200. At various known points along its course should be marked the levels above O. datum. For instance, the invert and cover of a manhole, the sill of a storm sewer, should be so treated. At the same time all storm sewers should be shown in blue instead of red; and to distinguish between the levels marked, they must be written in different colours according to what they denominate. For instance, surface levels must be black, sewer invert levels red, storm sewer invert levels blue. The foregoing reasoning is put into the form of a practical example in fig. 679. It is hardly necessary to add that all house

main to another manhole some distance away, in which is a steel cylinder called the forcing cylinder, similar in most respects to the air cylinder. This has previously been filled with sewage in a way similar to Shone's ejector, fig. 588. The air pressure on the sewage closes the inlet valve, and by its pressure on the sewage raises it to the intercepting sewer, which is about midway between the two manholes referred to.* The rising main is preferably made of cast iron, although it will not be under very much pressure. In the meantime, the



Excavation.

FIG. 670.

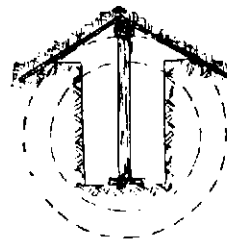


FIG. 671.

sewage in the air cylinder has risen to the top of a long-leg syphon, and thereby drawn itself off and flowed away by gravitation to the intercepting sewer. The process is then repeated.

The apparatus has the great advantage of being fairly simple and quite positive in action, and seems to have given satisfactory results. But of course the minimum flow from the high level should be quite equal to the maximum from the low level for efficient working. The flush tank need not of necessity

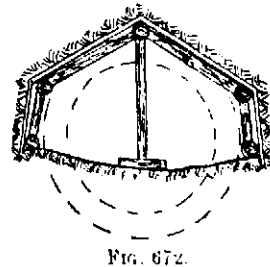


FIG. 672.

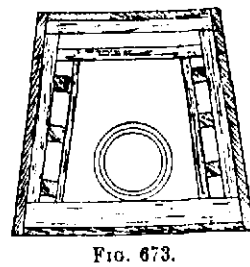
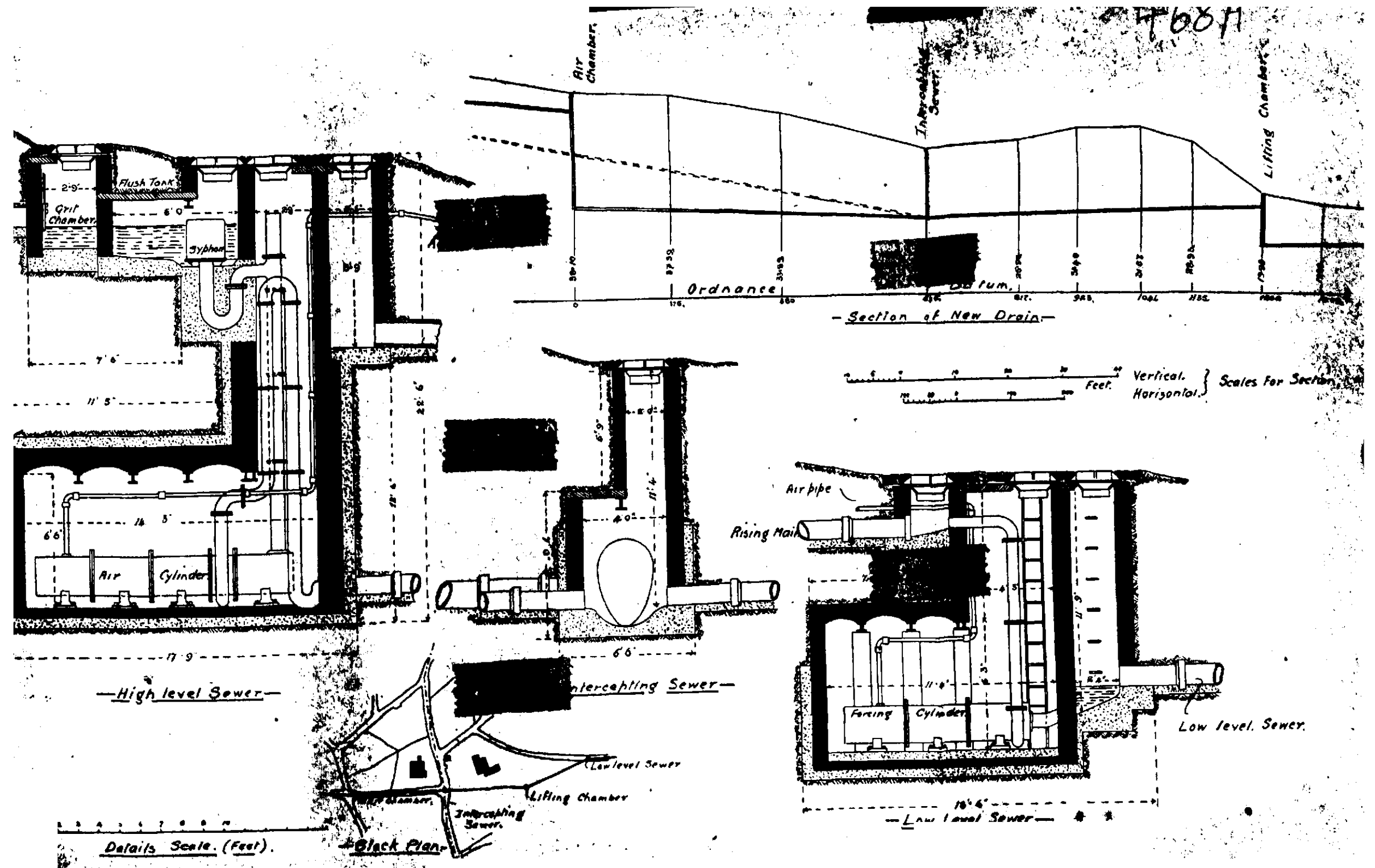


FIG. 673.

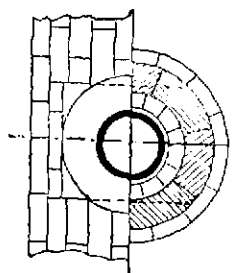
be contiguous to the air cylinder, but it must be connected thereto by a cast-iron pressure pipe. When designing, it must be borne in mind that the—

1. Air pipe must be higher than the syphon pipe. This is to prevent sewage in the air cylinder from finding its way into the air pipe.
2. The rising main must start at the bottom of the forcing cylinder.
3. The air pipe must enter at the top, and preferably at the opposite side.
4. The size of the flush tank must allow of the complete filling of the air cylinder and two syphon pipes + a few extra gallons.
5. The height the water can be pumped is limited by the height of the air pipe above the cylinder.

* In the system illustrated it is a gravitational sewer.



drains must be periodically inspected before filling in by the borough engineer or his assistant. They must be properly planned in accordance with his instructions (and he usually has great power in the matter), and tested by water and smoke before and after filling in, while the junctions with the main should be made by the local authorities' workmen, and the necessary work carried up to the boundary of the property, being afterwards charged for, or, as is usually the case, a deposit is made in the clerk's office, who instructs the engineer to



- PLAN -

FIG. 681.

do the work, that officer afterwards sending a bill of the actual costs in, when any balance is refunded. Although their use is now not much in favour, details of a lamp hole for built sewer are given in figs. 680 and 681.

In preparing a sewerage scheme for the approval of the Local Government Board it will be necessary to provide plans on tracing-cloth or prints mounted on linen showing the positions and sizes of the proposed sewers, manholes, ventilators, flushing-chambers, storm overflows (if any), outfall or sewage-disposal works. Existing sewers, with the sizes and levels, should also be given. The Ordnance plans are very suitable for this work, and can easily be brought up to date. A longitudinal section to the same horizontal scale,

and 10 ft. or 20 ft. to an inch for the vertical scale, and details of the manholes and other works to a scale of $\frac{1}{4}$ in. or $\frac{1}{2}$ in. to a foot, should also accompany the plan and section.

A detailed estimate on a Local Government Board form giving the lengths of the sewers, number of manholes, flushing chambers, ventilators, storm overflows, and junctions, and a brief specification, must also be given. This estimate should provide a sum for contingencies, generally 10 per cent., or the estimates should be made on a liberal scale. Nothing gives greater annoyance to a council or does greater harm to the reputation of the engineer, than to find it necessary to make an application for a supplementary loan to cover extras.

PLATE—MANHOLES AND FLUSHING TANKS.

1. Manhole with Three-Quarter Flushing Gate.
2. Manhole for use on gradients.
3. Flushing Manhole actuated by hand.
4. Automatic Flushing Tank.
5. Cast-iron Manhole for water-logged ground.
6. Manhole with sole entrance.
7. Manhole with side entrance, Section A B.
8. Manhole with side entrance, Plan.

CHAPTER XXVII.

SEWAGE DISPOSAL.

We shall now proceed to describe to the reader modern methods of sewage disposal for inland towns and places where it is not deemed advisable to allow crude sewage to run direct out to sea. We assume the reader to have a fair idea of how a sewerage scheme would be carried out, but we do not wish him to imagine that if he has not had some experience in the class of work that the few words on the subject in this book, or the greater details in other books, will make him a sanitary engineer. It will be assumed that he has calculated upon bringing six times the average dry-weather flow to the outfall works, where three volumes must be fully treated and three volumes partially treated according to L.G.B. requirements. The treatment adopted will then have to be decided upon according to circumstances.

The systems divide themselves into—

1. Land treatment only (sewage farms).
2. Chemical treatment, with final filtration or land treatment.
3. Bacterial treatment.

General Remarks.—Roughly speaking, sewage contains about 2% of nitrogen. It is this uniting with the dead organic matter which gives sewage its most offensive quality. The albumen is the principal nitrogenous compound. Inorganic matter in suspension is easily removed by precipitation, and usually gives no trouble. Soluble impurities, on the other hand, demand various forms of treatment. The nitrogenous matter must form nitrate when complete purification is aimed at, and carbonate of lime is the most suitable compound for effecting this, because it has an alkaline base. The object of introducing bacteria is to break up the organic matter into inorganic compounds. The toxine of the bacteria must be kept below a certain limit, and to do so a continuous flow of sewage through the tanks is essential. When sewage is first applied the bacteria will multiply very rapidly, and after a time this vegetative stage is overcome by a period of bacterial activity, when the tank is said to be "ripe." Bacteria themselves are resolved into two classes, Aerobic and Anaerobic, and during the process of sewage purification the aerobics exhaust the oxygen and prepare the way for the anaerobics, and finally the aerobics come back and reduce the nitrites into nitrates. In a chemical analysis of sewage it is necessary to find out—

First, the amount of organic or inorganic matter in the sewage.

Secondly, the stages of reduction or putrefaction the organic matter has undergone. These tests, usually placed in the hands of experienced chemists, are done by chemicals which determine the amount of free ammonia or buminoid ammonia in suspension or solution, the total solids, nitrates, and

oxygen absorbed. Neither odour or colour should be present in a sewage effluent, except perhaps in the case of trade wastes being treated; any turbidity shows a quantity of finely disseminated matter in suspension. The acidity or alkalinity should be ascertained, because a poor effluent or poor nitrification would be caused by acid which should be nullified. When any sewage enters and goes through a tank the following processes take place generally. After passing the detritus pit, and leaving all heavy refuse behind, the scum tank is reached. Here the heavy sludge settles to the bottom, and the oxygen in the sewage causes the aerobic bacteria to form the scum on the top. The scum will prevent any more oxygen from entering the sewage, and so below this anaerobic bacteria attack the nitrogenous matter and reduce it, together with the carbonaceous matter, to simple compounds for the aerobics to finally reduce. Bubbles will now generate in the sludge, and these carry up to the surface small particles of solids, which, again, will sink down to the bottom, and this is what is continually going on in a septic tank. There is really not much to choose between open and covered tanks (except expense), and in very cold climates a cover will effectually prevent the entrance of snow or the formation of ice, as the natural scum will suffice to shut out any light which is inimical to bacterial life, and hence the scum will form somewhat thicker in a closed tank than an open one.

For actual precipitations, either lime, alum, salts of iron, or other chemicals are used. The fine particles of lime or the flocculent particles of alum and iron, by their greater specific density, will assist the purification by causing a natural mechanical precipitation; and although this is only a first stage of purification, it will be the means of reducing two-thirds of the actual impurities. The tanks usually work on the continuous system. We shall now take the various systems in order and briefly describe them to the reader.

1. *Broad Irrigation.*—When the system is introduced the L.G.B. will require the purchase of one acre of land for every 100 persons supplying the drainage area. The system does not require any tanks; on the other hand, they are very often provided to assist in screening, reduction of some of the solid matter, and storing up the night flow. On the whole, they are a distinct advantage. It is of the utmost importance that the land selected shall be of a suitable character for the purpose; that is to say, it must be a loose loamy soil about 5 or 6 ft. deep. But even if clay were found to be present below the surface, it might be made very satisfactory by a proper system of sub-drainage. They must, however, be both allowed periods of rest. Some clays are sandy and slightly porous; others will consist of a sort of alluvial mud deposit, which will melt when in contact with water. Again, we may have very stiff and impervious clay. In this system the water does not penetrate very deeply into the ground, but will pass over the soil in thin layers, thereby taking up the oxygen from the air, the ammonia compounds being changed by the plant life into nitrates, a chemical change which, however, will take place in the dark below the surface.

If the soil be not very porous, sub-drainage has to be provided. The pipes must be laid in regular lines, at regular gradients, to inspection chambers, where the effluent can be seen, and if necessary sampled; then, if the engineer finds it has not yet attained the desired amount of purification, it must be turned into another drain, which by gravitation will bring it on to the surface of a field lower down for more purification. The stipulation of 1 acre per 100 persons is not always adequate. The best results would doubtless be attained by dividing the farm into 3 acres, one having a whole

year's rest, the second being used intermittently as required, while the third is worked up to its full capacity. Of course this quantity of land, when large towns are considered, is often not readily obtainable. It is the practice to sow such seeds as Italian rye grass on the farms, or lighter crops, such as wheat and barley, can be produced with advantage. The actual quantity of sewage any particular land will treat, of course, will vary: 200 ins. per year is perhaps about the average on each acre. When the land is stiff clay it will crack in hot weather if allowed to get dry; it is therefore essential to discharge on to such land slowly and at a uniform rate, but if this is impossible a surface coating of clean ashes 15 in. deep will prove effective.

Again, when the sub-drainage which we advise is made so as the drains are too close, the sewage will not get enough filtration, and a growth of sewage fungus will set up in the pipes. The cells of this fungus contain much free sulphur, which will give rise to hydrogen sulphide and much smell. The land



FIG. 682.

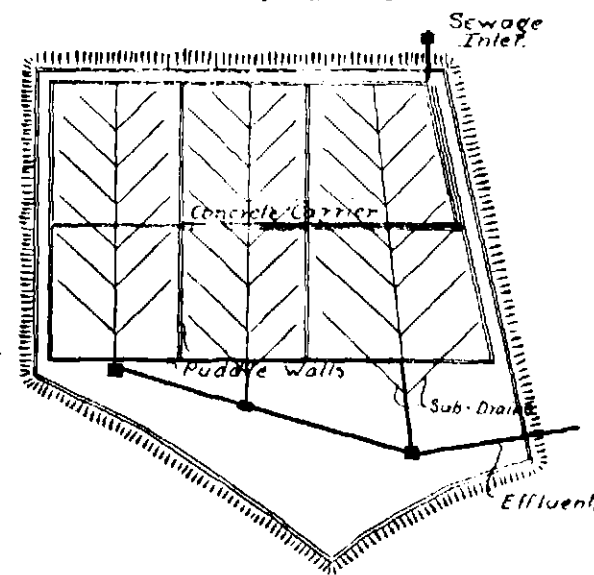
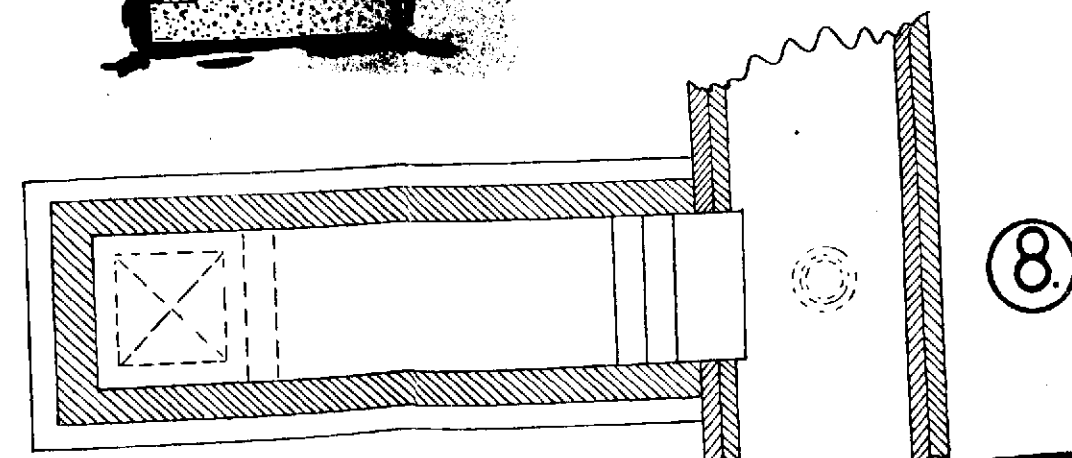
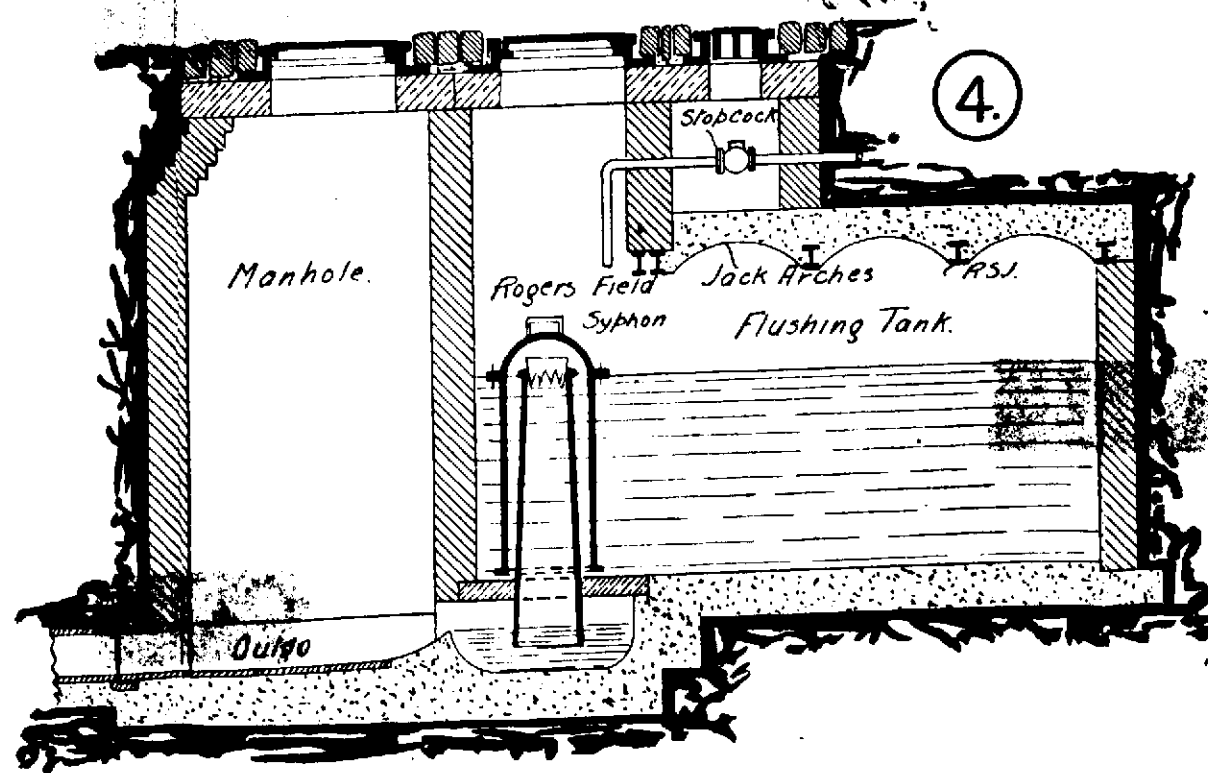
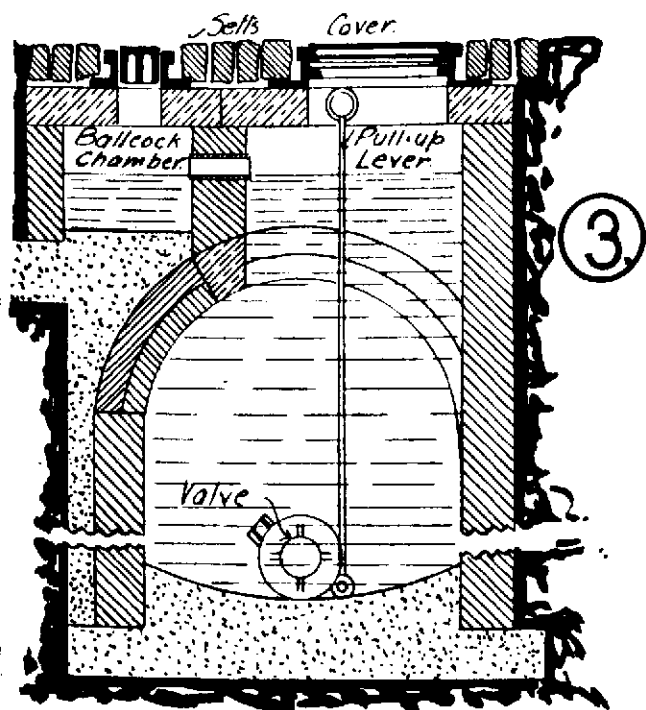
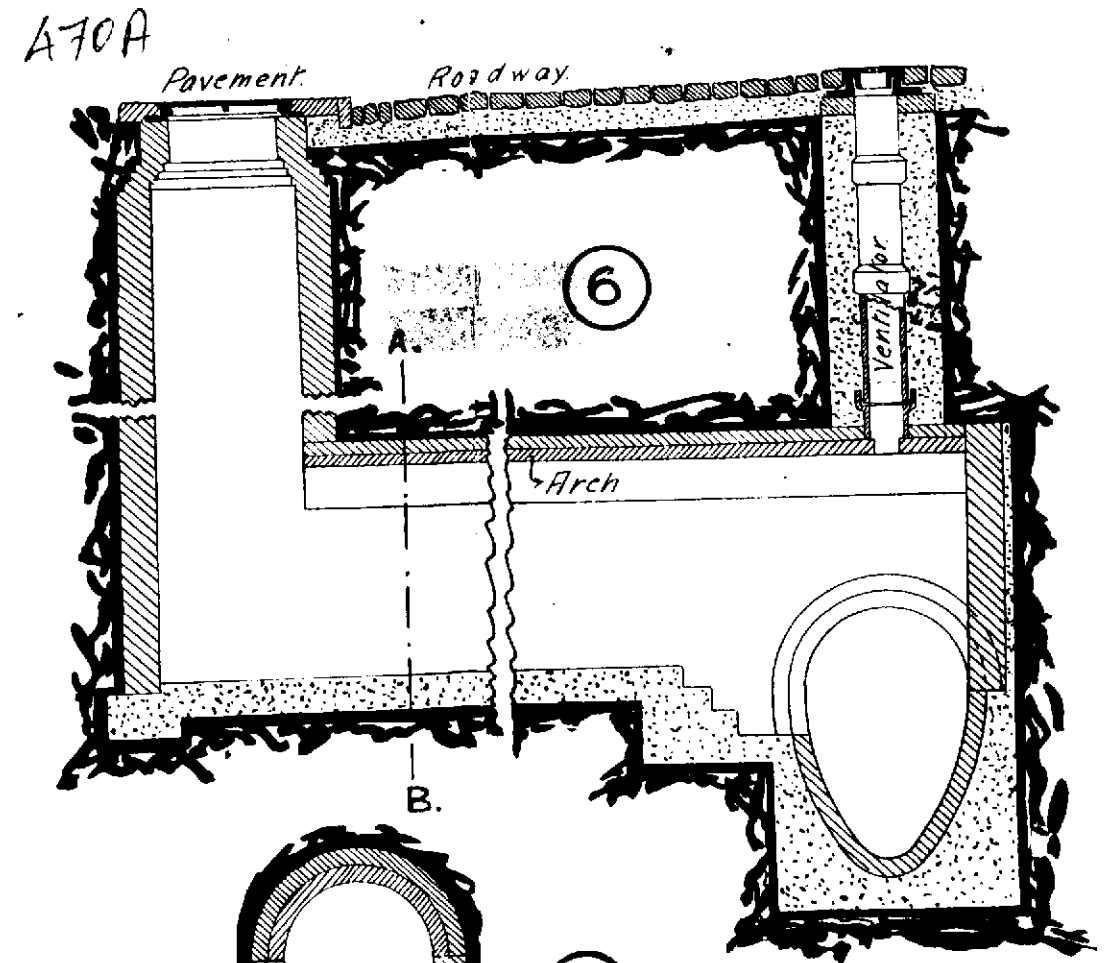
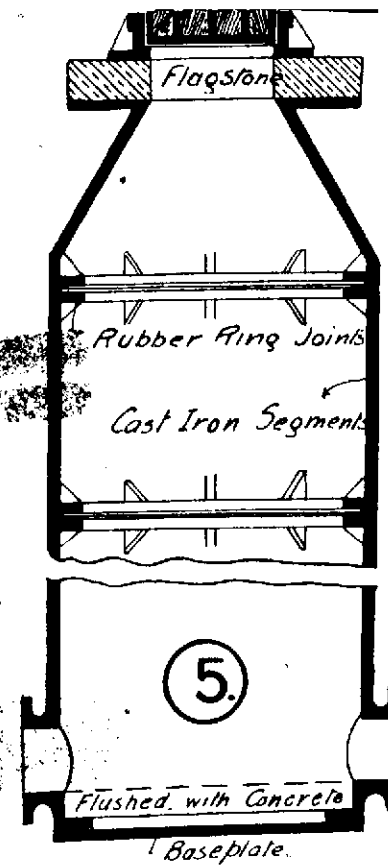
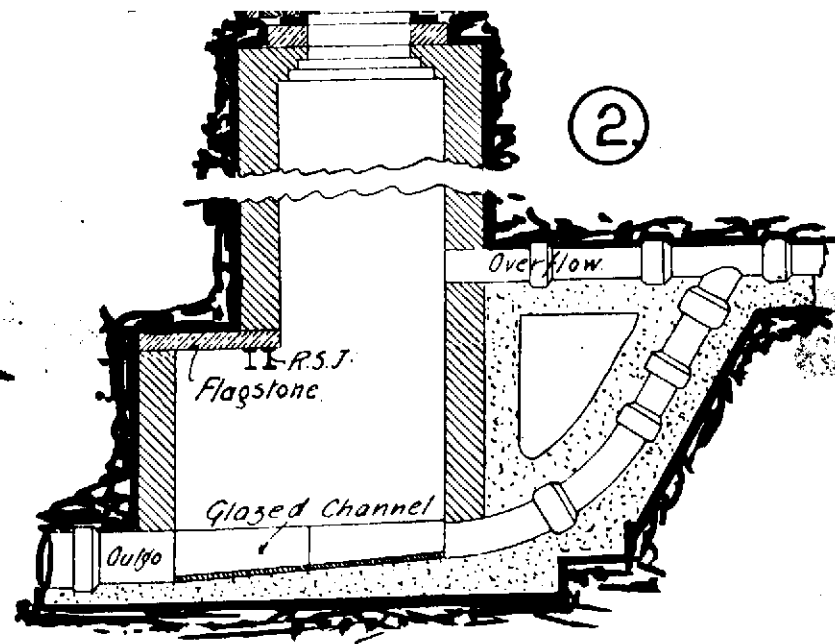
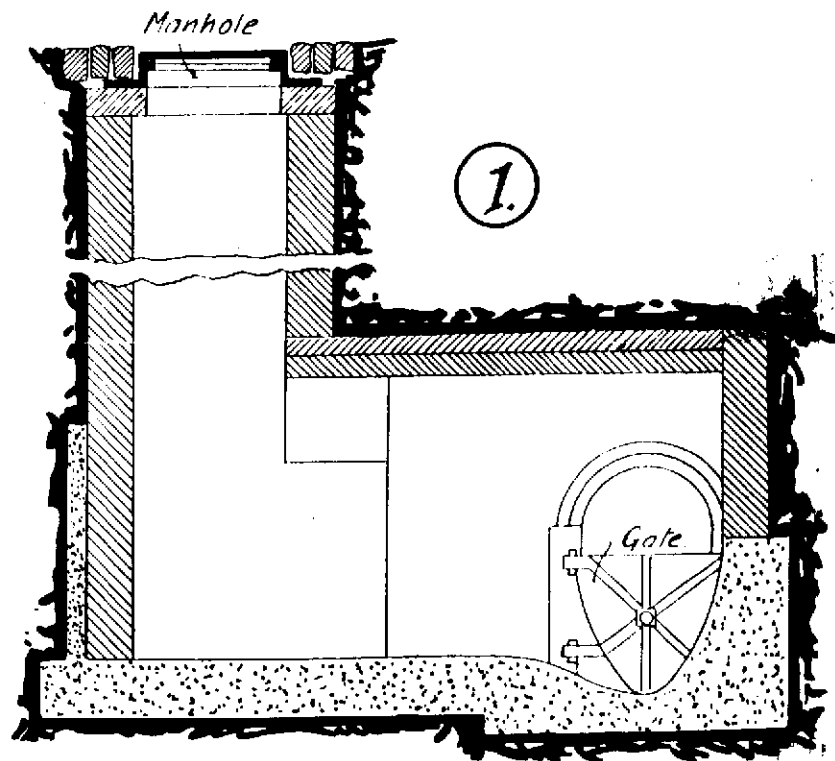


FIG. 683.

must also have a natural slope; about 1-80 is advisable. The main conduits would be of concrete or brick. They must be of ample size and very flat in the gradient, to get which they are provided usually with overfalls; these also assist in the purification by aerating the sewage. The outlets would be above the level of the main flow, the sewage being dammed back by weirs. Either of these outlets discharge into what is termed technically a *main grip*. They are trenches about 9 in. wide x 6 in. deep, and at every 50 yards or so have stop plates of wood or iron, as in fig. 682. This causes the liquid to flow over the land in thin sheets. The sub-drains would be formed of 4-in. field drains laid anything up to 1 chain apart according to circumstances (*vide Land Drainage*). But the main outfall pipes should be cement-jointed in the usual way. The above system of purification is shown in a practical form in figs. 683 and 684.



The Chemical Treatment of Sewage.—When chemicals are chosen for precipitation, it is absolutely essential to choose chemicals which suit the composition of the sewage they are going to purify. From the outset we wish to firmly impress upon the mind of the engineer that our remarks upon the disposal of sewage are only of a general nature; that is to say, the most successful sewage scheme of one town will be entirely unsuitable in another because of the difference in composition of the sewage and general local circumstances. That is to say, if he sees a very nice-looking and eminently satisfactory sewage scheme in town A, he must not suppose he can lay down the same for town B without any further consideration whatsoever. Great care has to be taken to avoid secondary decomposition. The services of an experienced chemist are absolutely essential in the first instance. Chemical

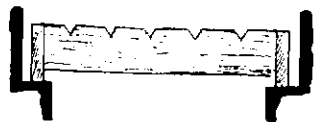


FIG. 688.

precipitation will remove from 80 per cent. to 90 per cent. of the solids, but not so the albuminoid ammonia. The sewage must be screened, vertical screens being usually employed. They should be removable, and accessible for cleansing. Continuous revolving screens are also an advantage. The chemicals must be properly mixed; and if a liquid state is required, Kierby's mixer seems to have given very good results. Another and simpler way, very effective also, is a V-shaped trough placed across the main channel, and always full up to the small notches. The idea is shown by fig. 688. The following are the systems of chemical treatment in vogue:—

1. The A.B.C. process (using alum, blood, and clay).
2. Lime.
3. Sulphate of iron.
4. Sulphate of alumina.
5. Perchloride of iron.
6. Alumino ferric, and the more or less fancy processes of magnetite, hermite, oxygen, and electrolysis.

The A.B.C. process did not survive very long. Charcoal is used in place of the blood.

The lime process is used in many places. It is reduced to milk of lime, and the quantity required varies from 6–12 grains per gallon. The process makes very good manure, but the effluent must be filtered or have land treatment to reduce its alkalinity, which is detrimental to fish life. It produces a very heavy sludge naturally, as the lime itself all goes to make sludge. This, however, may be successfully reduced by adding 3 grains of sulphate of alumina or 1 grain of sulphate of iron. This latter chemical is obtained in crystals, and has to be dissolved in lead-lined vessels. The alumino-ferric treatment is fairly simple. A cage which contains a large block of the alumino-ferric is hung in the main flow of the sewage and is dissolved as required.

The International Company's system of purification is also worthy of attention. The principal precipitant is ferrozene (also called polarite). It is magnetic ferrous carbon. Polarite, however, is quite insoluble, consisting of magnetic oxide of iron, and is carbonised in retorts. It will absorb lead and iron from water, and will give off much oxygen. It therefore becomes a very powerful deodorant. The effluent from this system passes to clarifiers, and thence to specially constructed polarite filters, ventilators being provided.

Electrolysis.—This is a system of sewage purification which is only in

its experimental stages, but it bids fair to give good practical results when has had the attention of experienced chemists. The treatment consists of series of plates of cast iron placed in the flow of the sewage, and connected the positive and negative terminals of a dynamo. The electric current itting up the constituents of the sewage, the soluble organic matter is dised and burnt up by the nascent oxygen. Oxides of chlorine are then olved. The chlorine and oxygen combine with the plates of the positive le. The system has the advantage of producing very little sludge, and the uent is practically pure after passing the settling-tanks, and filters are not ially required. Electric filters could be provided, doubtless, layers of ce being used as electrodes.

Bacterial Treatment.—Although not actually so, there is no doubt that ad irrigation and intermittent downward filtration are systems of bacterial atment of sewage. Perhaps one of the best systems of bacterial sewage atment is the Scott-Moncrieff system. It consists of the first and second tivation tanks and an automatic filter. The system, no doubt, is thoroughly entific; the screening system of the first cultivation tank allows the heavy neral solids to subside, while the second tank allows very complete aeration; the other hand, it will require a large head of sewage to work the system.

bacterial treatment of sewage the bacteria convert the organic matter oipally into ammonia, marsh gas, and nitrate. Each phase of the breaking-wn process is carried out by different bacteria, while the final stage consists nitrification, in that the ammonia is first reduced to nitrous acid (by dation), with further oxidation to nitrates. The presence of chemicals and nufacturers' waste will, to a great extent, spoil the effects of bacterial atment (with perhaps the exception of permanganate of potash), and the fall must not be too acid nor too alkaline. It is usual to provide two beds, rimary and secondary, the former consisting of 2-in. to 3-in. stones, and the ter ½-in. to ¾-in. gravel. The primary bed acts on the larger solids to be rated on by the organisms, the secondary bed permits of complete oxidation. e beds, no matter how well made, do no doubt lose efficiency in time, due wear and natural settlement. Regarding tank capacity, the following are tracts from the L.G.B. requirements:—

- (1) Two volumes of storm water plus the normal sewage flow must be ited as sewage proper.
- (2) An additional three volumes have either land or filter-bed treatment.
- (3) If the separate system is adopted, one volume of storm water + normal r is treated as sewage proper.
- (4) Three fillings per day of twenty-four hours is allowed.

From the above conditions we can establish three equations whereby we can l the capacity of the primary, secondary, and storm-water beds, provided re are also sedimentation tanks.

Given a = water supply in gallons per head per day.

" b = depth of filters (4 ft. 6 in. recommended).

The superficial area of the primary bed and the secondary bed will be in

$$= \frac{01105a}{b} \quad (396)$$

the storm-water bed

$$= 00124a \quad (397)$$

If trade refuse is allowed into the sewers, these figures will have to be

A good inexpensive form of carrier is formed by half pipes carefully laid and cement-jointed. The following results may be easily expected on such a filtration area.

	Solids in Suspension.	Solids in Solution.	Oxygen required to oxidise.	Free Ammonia.
Crude Sewage	31.80	47.15	1.99	2.43
Effluent	0.00	30.61	0.534	0.314

2. *Intermittent Downward Filtration.*—This system requires loose sandy soil about 6 ft. deep, but the Local Government Board allow 1000 persons to drain on to 1 acre. The ground must be in level plots with ridges 5-6 in. apart and furrows 20 in. deep. The ends of these are stopped and the sewage is allowed to fill these natural "tanks" to about 6 in. deep. It is then allowed to filter away, and the ground left to aerate for several hours.

Tanks.—The entry of sewage into the tanks is usually over a weir which runs the whole width of the tank, and there are sometimes baffle plates therein to cause the liquid to take a circuitous course. But a system shown by fig. 685 is also much used and appears to give satisfactory results, by causing



FIG. 684.



FIG. 685.

a certain amount of upward flow of the sewage and hastening the subsidence. Resting tanks are not now much used, the continuous system having been found to give more satisfactory results. The maximum flow in these tanks, however, should not be more than about $1\frac{1}{2}$ ft. per minute. To deprive the sewage of any floating matter, planks on edge are provided; and instead of the baffle plates proposed, it may be deemed more satisfactory to form cross walls in the tanks to effect the same purpose. The construction of sewage tanks does not demand any great skill; the local circumstances will guide the engineer. They are usually of concrete, lined inside with neat cement, or they may be brick-work in cement, plastered inside, or the sides may be lined with Stafford blue bricks. They must be designed according to the principles set forth under "Walls," to withstand the inside water pressure when full, and the earth pressure outside when empty. Any sills over which much liquid passes should be of dressed granite or other hard stone, carefully pointed, and, if necessary, dowelled to each other. The removal of sludge from the tanks demands attention. In some cases it can be removed continually by the pressure of the liquid above it. In other cases it would be necessary to empty the tanks which have a natural fall to the sludge valve, and the sludge swept by hand into the main sludge channel, whence it flows away by gravity to the sludge pits for final removal.

Another ingenious device is that shown by fig. 686. Here we have an iron tube parallel to the bottom of the tank and about 2 in. above it. It runs

on two tram rails, which are fixed to the bottom of the tank, by means of two flanged wheels. The pipe is perforated, and attached to it is a rubber squeegee. By means of winches the apparatus is drawn along the tank and brings with it the sludge. At the top is a pump which exhausts the air from the pipe, when syphonic action will cause the sludge to come over into the

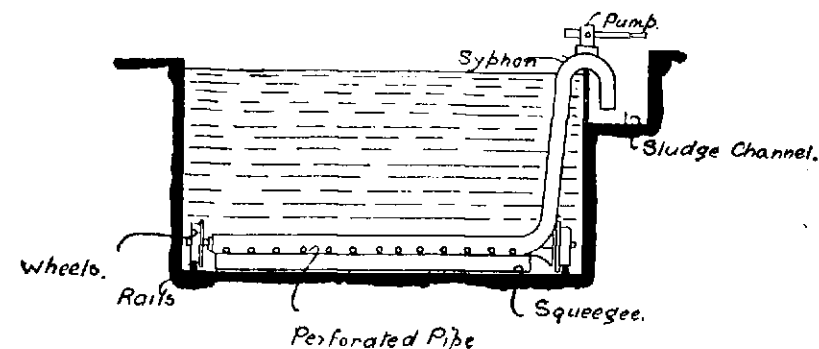


FIG. 686.

sludge channel, as shown in the diagram. Those tanks of the circular type have revolving scrapers on the same principle. The modern tendency is, however, gravitation removal into a main sludge channel, and raising it if necessary by pumps. There are also various patent forms of tanks, notably the Dortmund tank. They all have more or less of the qualities which they

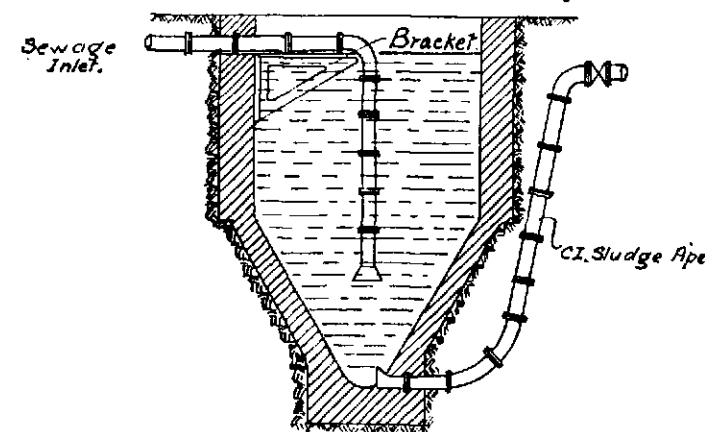


FIG. 687.

claim, but are more or less of a fancy nature, which is to their detriment. The Dortmund tank has, however, given some success in the past, and consists of a conical bottom and vertical sides. It is circular in shape and about 20 ft. deep; a cast-iron bell-mouth pipe hung in the centre discharges the sewage near the bottom of the tank, while the effluent channel may be hung on a bracket from the side. A diagram of a Dortmund tank appears in fig. 687. We shall give some practical examples of sewage works later.

modified, so that instead of putting down merely "*a*" we must find the quantity of trade refuse that enters and reduce it to gallons per head per day and call its value *c*, then instead of *a* simply we shall use the value

$$a_1 = \left\{ (a+c) + \frac{a+c}{10} \right\} \quad (398)$$

The cost of renewing and keeping the beds in order would be about 5d. per square yard per annum. Fortunately trade refuse is not usually present in town sewage in such quantities as to *prevent* bacterial treatment, nor is its quality likely to have any very specially detrimental effects. Neither has the formation of ice on the surface of the beds been proved to be detrimental.

The question of allowing trade refuse into the sewers requires careful consideration, especially if there are easy and practical ways of otherwise getting rid of it. It will naturally render the treatment of the sewage more difficult, and may injure the brickwork of the sewers, or by its density choke the sewers and cause very bad gases. Chemicals would not be objected to provided they were not too acid or too alkaline, but the compounds of iron and chlorine have been found to upset bacterial treatment. Pumping expenses would also be increased, as against the ordinary 6 volumes of sewage we have to provide for 6.6 when we are dealing with manufacturers' waste.

A river in a town which has a combined system of sewerage will naturally be much freer from any pollution than one upon the separate system, but, again, a rain storm, when heavy and sudden, will not by any means always dilute the sewage to safe limits, but may, on the other hand, bring down large quantities of suspended organic matter, and the state of the sewage coming into the rivers by the storm overflows varies, because the storm might come down at the time of the minimum dry-weather flow, hence great precaution is necessary in drinking river-water, the presence of .002 grains of albuminoid ammonia being dangerous, and micro-organisms must not be present in a larger quantity than 25 per cubic centimetre.

Rivers themselves are natural purifiers, but the actual degree of purification depends upon the depth, velocity, and physical character. The purifying actions consists of oxidation, natural deposition, and by the action of micro-organisms. Weirs and falls which cause aeration will assist the oxidation. It is the light flocculent matter which is the last to subside and the first to putrefy.

In large sewage schemes the disposal of the sludge is often a very difficult and serious matter. With a seaboard city of any magnitude, or any city which has reasonable access to the sea, it is best got rid of entirely by taking it away in specially constructed boats and dropped away out at sea, far out of reach of any harm. The practice is carried out with success in London, Manchester, Salford, and Dublin. One might expect a return of some sort from sewage sludge, but, unfortunately, owing principally to the water difficulty, the process of manufacturing it into forms of manure is not satisfactory; in some places, in fact, farmers will refuse to have it at any price, or no price at all. Sludge usually contains 90-95 per cent. of water, and under chemical precipitation it becomes of considerable quantity, but septic tank treatment will tend to reduce it greatly. Of course it can be buried, but the amount of land required to do so very often prohibits the procedure. It has been burnt, after air-drying, with more or less success. But where it cannot be buried or taken to sea, pressing under heavy pressure has been found to be the most satisfactory, which rids it of a great percentage of its

moisture. The cost will be about 2s. per ton of pressed cake. This cake has not much tendency to putrefy, and some farmers will remove it, but more often than not it has to be burnt. In greasy sewage the removal of the grease from the sludge is a fairly profitable procedure, especially when it is of a quality suitable for soapmakers. To be of any real profitable value the sludge must yield 5 per cent. of grease, selling at £9 per ton.

When taking samples of sewage for analysis in connection with matters pertaining to river pollution, extreme care must be exercised, because in order to estimate the composition of any given sewage, samples of it must be taken at least every hour for twenty-four hours, and the samples so taken thoroughly well mixed in direct proportion to the amount flowing at each collection, which is preferably ascertained by a gauge similar to fig. 792; because if the same quantity of sewage is taken from a sewer running nearly empty and nearly full and mixed, the resulting mixture will not represent the average composition of the sewage at the two periods in question, and it is by reason of this very important fact having been neglected by the responsible parties that old records are not to be placed too much faith in. When the sampling is properly done, the averages will show something like 72 parts of solid matter in solution and 45 parts of suspended matter in 100,000, the matter in solution being composed of 4.69 parts of organic carbon, 2.20 of organic nitrogen, 6.70 of ammonia, 7.72 parts of combined nitrogen, of which a very small amount occurs in the form of nitrates and nitrites, the matters in solution containing 44.69 of suspended matter, consisting of 24.18 mineral and 20.51 organic. Such results are given as the average in the First Report of the Rivers Pollution Commissioners, but great variation may be found in exceptional instances, and it must not be supposed that sewage contains mainly human refuse. Kitchen refuse, stables, cow-houses, and slaughter-houses all contribute their share.

The injurious effects of sewage discharging into rivers are numerous. Of the rivers in the North of England, and especially Lancashire and Yorkshire, many are little better than sewers even now: the river Irwell in Manchester being the worst case the writer has yet seen. The result is that the emanations of such rivers are distinctly prejudicial to the health of the community who live near them; and not only is their health affected, but their general well-being and prosperity. For farmers, for instance, the water is unfit for cattle, householders cannot let their houses, and landowners can only sell the land at prices which bring no return. This, however, is not all. There are scores of cases where sewage-polluted river-water is used for drinking purposes, which, even if it does not directly cause outbreaks of disease, must be bad for the general health.

The idea put forward by some, that the flowing down the channel for a few miles by the polluted water will purify it, is erroneous. There is not a single river in the United Kingdom capable of doing this, for the simple reason that they are too short. Once contaminated, water is undoubtedly unfit for human consumption. It is a fact, however, that the nitrogenous matter in sewage does to a great extent decompose during the passage of the sewage through the sewers, but after entry into the river or water course it is very slow. In fact, a flow of 11 or 12 miles has been shown to effect little or no alteration on the organic matter dissolved in the water, what alteration there is being due to the oxidation of the organic matter by the dissolved oxygen contained in the water, which has a greater effect than gaseous oxygen, but even then there is less than usually supposed. It is the foul organic matters in solution which are so very

persistent. They oxidise very slowly and are only very partially removed by sand filtration, and there is no single process of sewage purification yet known which will render the water of the stream into which the effluent flows really fit for subsequent potable purposes, because cholera and typhoid may be contracted from the infectious abdominal discharges of the patients. In a lesser degree also the same remarks apply to dysentery and diarrhoea.

There is no doubt, however, that sand filtration does improve matters in waterworks (although even in its most efficient state the invisible matters in suspension are still left in the water) by reason of removing suspended impurities and diminishing the proportion of organic matter in solution. The extent of this will depend mainly on the rate of filtration and the thickness of the filtering medium. In fact, sand filtration is the best known means of purifying water, but there is no evidence to show that pathogenic germs are destroyed.

So much for the aspect of river pollution as regards the upper (tideless) parts of the course. We now come to consider what is the result in tidal waters.

Some of the best evidence on this score is set forth in the First Report of the Royal Commission on the Metropolitan Sewage Discharge, 1884, in which it is stated that eleven and a half million of cubic feet of concentrated sewage enters the river Thames during three hours every tide. The main outfalls are at Barking and Crossness, on the north and south sides of the river respectively, the sewage being let out on the ebb tides and stored in tanks at other times. The solid matter in the sewage decomposes, and the river-water dissolves the gas given off, which, again, escapes when agitated by the passage of passing steamers.

The water was stated to be so foul as to be unfit for washing ships' decks. Lower down the river, however, the natural oxidation causes the nuisance to be somewhat abated, and below Greenwich it is stated to be inappreciable, which, as a matter of fact, is surprising, bearing in mind the enormous population of London, which would produce faeces in their natural state to the extent of about 600 tons daily. The same thing took place in the Liffey in Dublin until a few years ago, due to the sewage going direct and unpurified into the Liffey. The writer has known the smells on the quays to be not only disagreeable, but actually sickening and nauseating. It is, however, much to the credit of the city engineers that the new main drainage works have vastly improved this river, so much so that fish have been reported at Custom House Quay. This improvement is all the more manifested by the fact that during a temporary stoppage of the works in 1909, by silting up of the new outfall (which does not reflect such credit on the engineers, who, however, put the blame on the old sewers which discharge into it), the river in the space of a few days gave rise to complaints and discussion in the papers from all parties. There is no doubt that the discharge of unpurified sewage in the rivers injures and completely destroys the fish, due to the oxygen absorbed from the water.

Another aspect of the question is the silting up of the channel, and consequent hindrance to navigation: the Mersey, Clyde, and Avon have suffered from this cause, while it has been shown in one instance that, inland, the raising of the river bed has caused flooding of the surrounding country, evils which may be even increased by the alkaline constituents of the sewage decomposing and depositing the calcareous constituents of the water.

In tidal waters the effects of pollution are not confined to the channel below

the outfall. The same evils exist above it, having been proved in the Thames, by means of floats, that considerable oscillation of the sewage takes place due to the tides, especially during the change from spring tides to neap tides. The continual flow of fresh water, however, tends to somewhat mitigate this mixing action which will also take place due to the difference in the specific gravities of the liquids and the continual outward flow. It has been pointed out that the purifying effect of the passage of sewage down a river channel is due mostly to oxidation. It must be borne in mind, however, that this is not all. There always exist animals of a low type which feed upon the sewage, such as water-fleas, minute algae, and low plant life, the latter being found more especially in polluted back waters, and whose purifying effect consists in giving off oxygen in distinction to the water-fleas which consume the sewage.

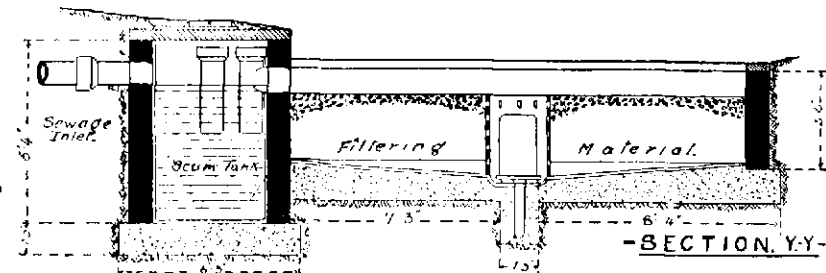


FIG. 689.

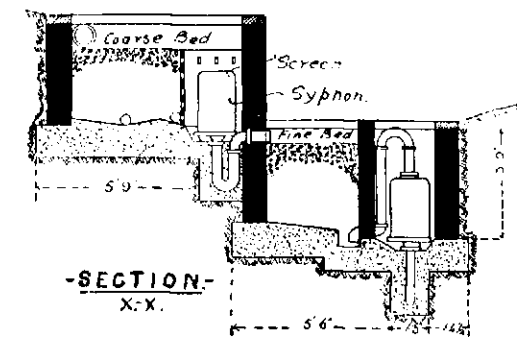


FIG. 690.

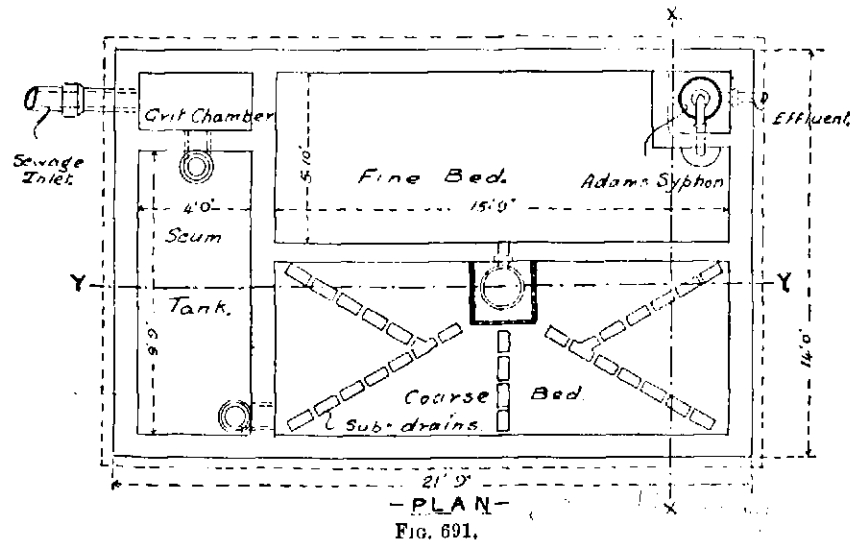
Summing up:—

The principal evils which arise out of letting crude sewage into tidal estuaries are—(a) The considerable length of time which it takes large volumes to actually reach the sea, especially in dry weather, due to oscillation.

(b) The various salts contained in sea-water cause considerable precipitation of organic matter, while at the same time they retard oxidation and purification, and hence in tidal estuaries we have that noticeable quantity of brown slimy mud deposit.

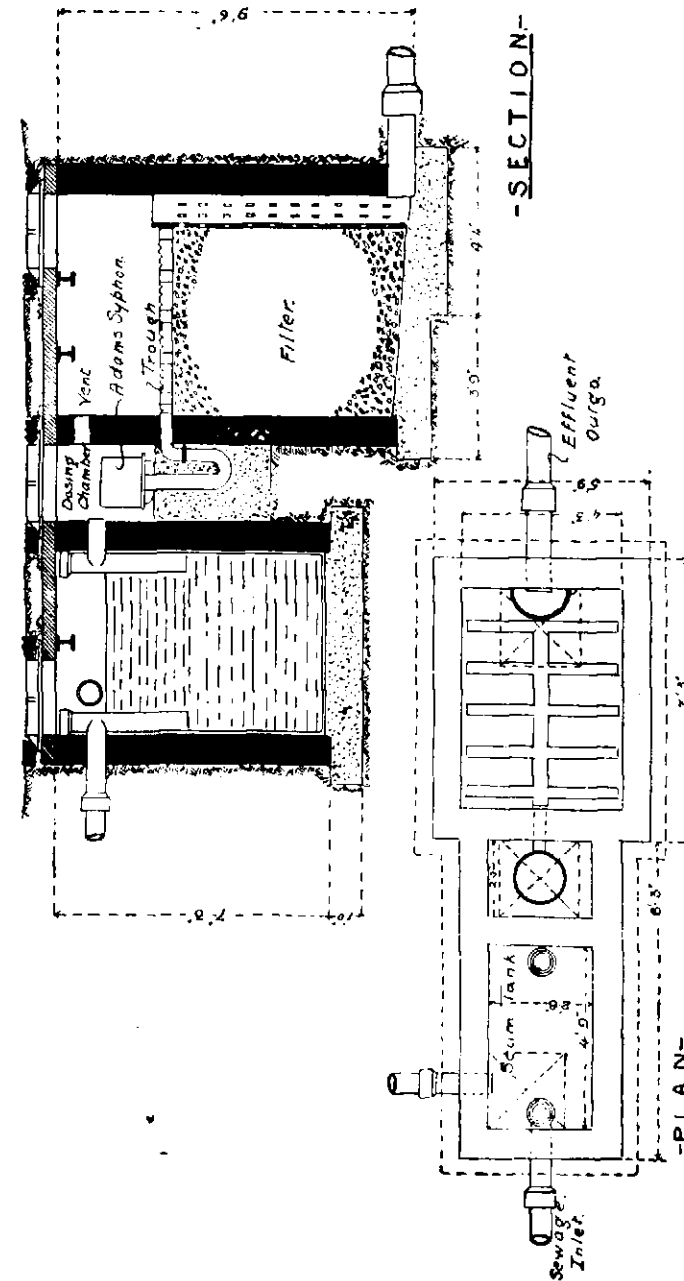
We shall now conclude the chapter with a few drawings of typical examples of sewage disposal works and the apparatus for the efficient working of the different processes of disposal and purification, and afterwards a description of a typical example of successful main drainage disposal works. In figs. 689, 690, and 691 is shown a type of tank and filter beds for the disposal of the

sewage of a small village or estate on a bacterial basis, in conjunction with Adams' automatic syphons. The action is simple. The sewage entering by the grit chamber, parts with the very heavy solids and foreign bodies, and flows over into the scum tank, where some of the sludge is disposed of. When full, the sewage slowly flows over the primary bed. Again, when this is full the automatic syphon comes into action and discharges, practically instantaneously, the whole contents on to the fine bed and floods it. When the flooding has subsided another syphon draws it off to the effluent. A smaller scheme and not so expensive is shown in figs. 692 and 693, in which the filter is closed, but freely ventilated. There is no secondary bed, and the contents of the bed flow away by gravitation. The system, though suited to small installations, is not as satisfactory as the one just described. By the courtesy of Messrs Mather & Platt, Salford Ironworks, Manchester, who have very valuable experience of all matters pertaining to sewage disposal



works, I am able to present to the reader some examples of modern practice of sewage disposal on a large scale. Fig. 694 and attendant details shows a general arrangement of a sewage scheme according to Local Government Board requirements. The plan was prepared to suit two schemes, and the dimensions are given under letters A and B. It will be seen that the sewage enters the detritus chamber, which is divided into two sections, both of which can be used at the same time, and the whole of the sewage can pass through one chamber while the other is being cleaned. Up to three times the dry-weather flow passes on to the sedimentation tanks, the quantity being discharged on to the open trough spreaders by means of a patent measuring valve. The remainder of the sewage over this quantity passes the overflow weir for storm water, and is discharged into the storm settling-tanks. It will be seen that there are three outlets from the storm settling-tanks, one of which is taken to the measuring chamber to allow up to six times the D.W.F. to be discharged on to the spreaders. And all sewage coming down over six times the D.W.F. passes on direct to the river or land as required.

Fig. 695 shows a sewage scheme consisting of two open trench spreaders working intermittently. They are both fed by one measuring valve; there



is also an open trough spreader, fed by a measuring valve, for the distribution of storm water. This plant is *not* in accordance with the Local Government Board Regulations, but in some instances it is necessary that such systems be adopted.

Spreader for Sewage Disposal Works.—The chief conditions of the distribution are—

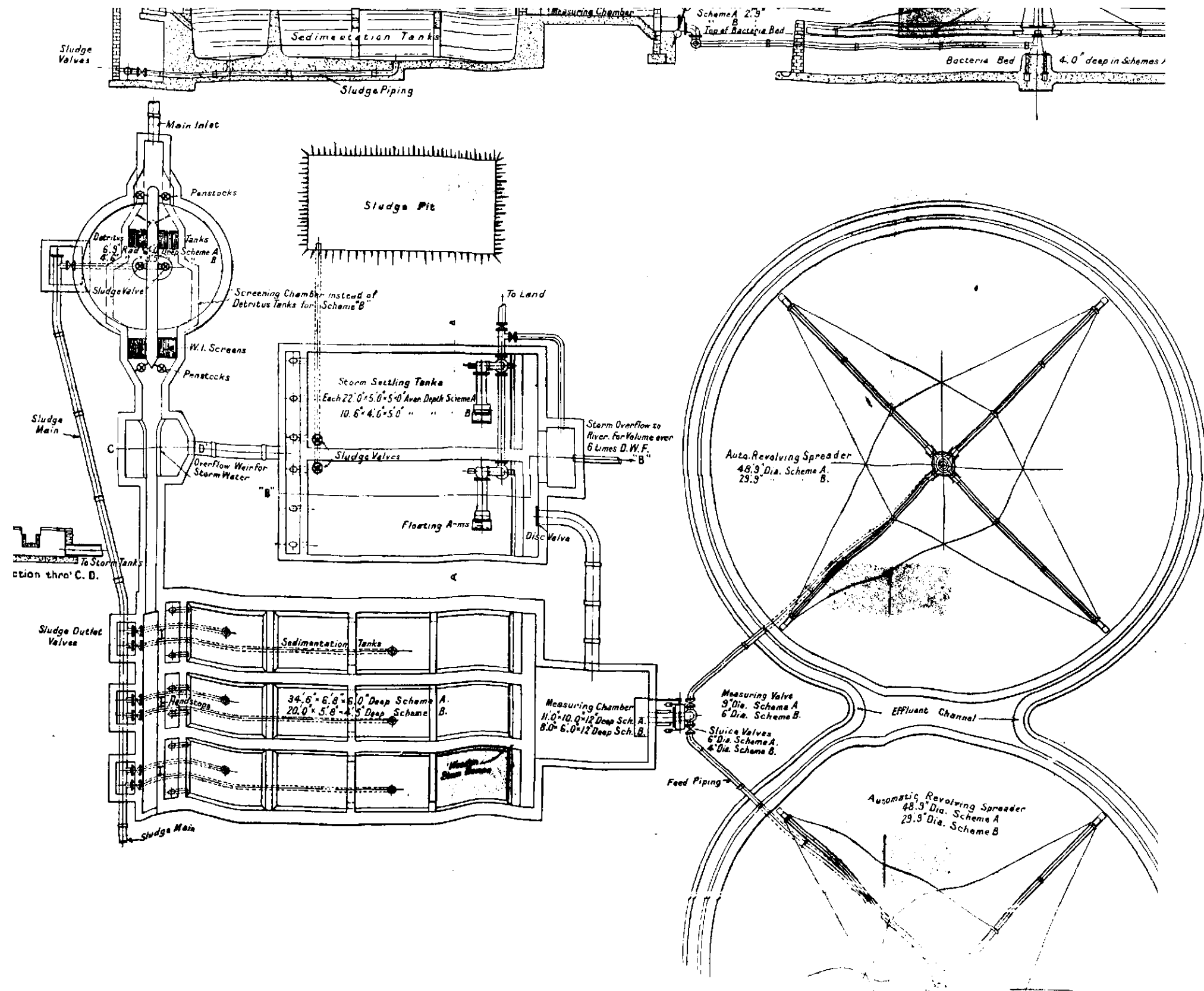
1. Uniform and fine distribution without exposing a large sewage area.
2. The slowest economical rate of travel.
3. Avoidance of freezing risks.

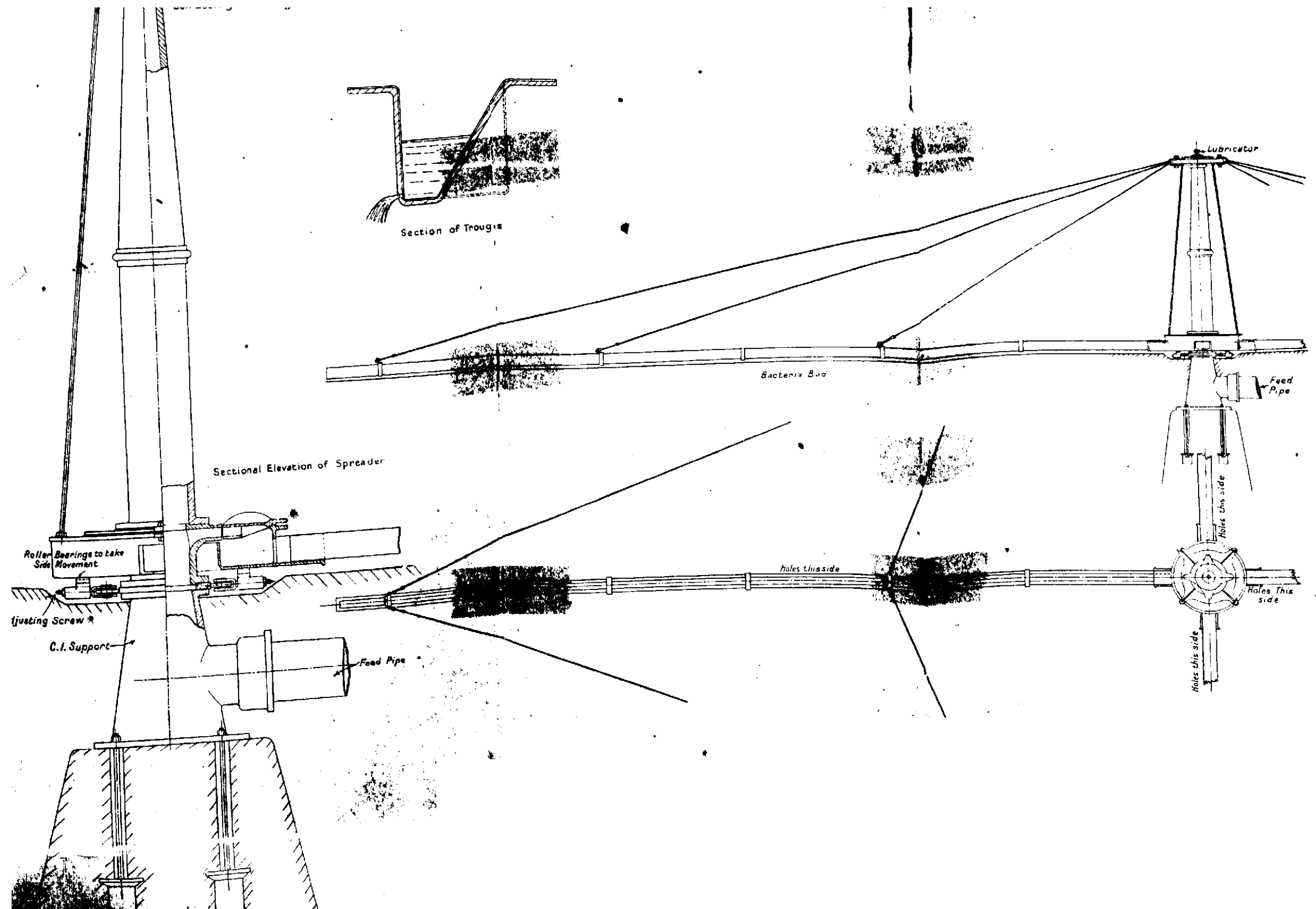
Uniform distribution is essential.

Ordinary perforated pipes have been used in some places with success. They want a fairly large head to work under, in order that a good spray may be obtained. Perhaps the most satisfactory form of spreader is the revolving type, of which many are in use at the present day. When the effluent is not good, however, when it comes from the tanks, but contains a large amount of suspended solids, those of the closed pipe type are very liable to choke, and due provision must be made for thorough cleansing. These difficulties are obviated in the open trough type. Fig. 696 shows details of such an open trough spreader as made by Messrs Mather & Platt, Manchester, and is to be well recommended. It has the important advantage that all liability to smell emanating from sewage remaining in the arms is eliminated, as the perforations are made at the bottom of the troughs, and the liquid being open to the atmosphere, assists in better oxidation thereof. Further, the centre of the spreader is fitted with special turbine guides, which assist in the rotation of the machine.

The pipe-arm type of spreader by the same firm is shown by fig. 697. The particular one in question is of the continuous type, the sewage passing up through the centre of the support, and overflows into a revolving chamber to which two of the arms are connected. Two subsidiary chambers are attached to the main chamber, and thus, when the sewage rises up to a certain level, it will overflow the weirs into these latter chambers, thus causing the arms attached thereto to come automatically into operation. This arrangement avoids the use of joints or seats, which are always more or less a source of trouble. Both sets of spreader referred to are fitted with two sets of ball bearings at the top cap, one to take the whole weight of the revolving portions of the spreader, the other to take up any side movement at the head. A set of roller bearings are fitted to take any side movement at the centre of the spreader, the rollers revolving around a special case-hardened steel ring fitted between the support and the flange of the centre; thus any wear which may take place is reduced to a minimum, and in course of time the ring can be renewed if required.

Regarding the disposal of sewage, no better information could be placed before the reader than the recent Report of the Royal Commission which was sent to report on the various methods employed throughout the country. The Report itself, consisting as it does of the opinions of nearly every municipal engineer in the British Isles of any standing, besides the opinions of various eminent chemists and other scientific men, is a veritable mine of information. However, it necessarily contains much which will not be of primary interest to the engineer; and, owing to its naturally large proportions, the writer has made a very careful abstract from the Report specially for the benefit of his readers, omitting all that which he deemed superfluous. No one man's opinion could be of such value as this Report; and therefore I have somewhat





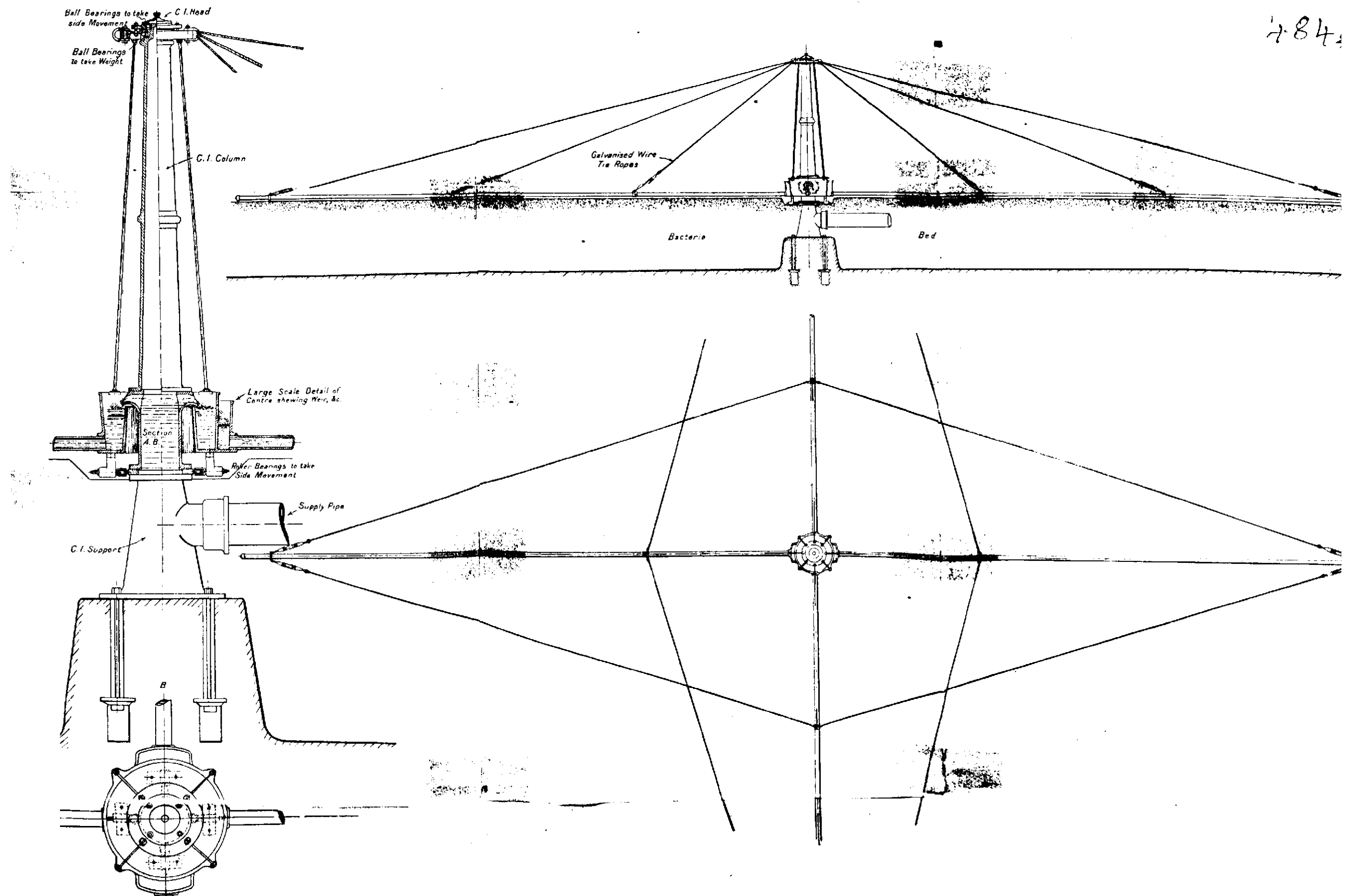
curtailed this chapter, and given the reader the benefit of the very latest information on a most important subject. In the chapter following the Report I have placed before the reader a detailed description of a large sewage scheme, that of the county borough of Salford, partly because it is a very good example, and partly because the author served his pupilage in the very sight of the same works. I am indebted to the Borough Surveyor, Mr J. Corbett, for information kindly laid at my disposal.

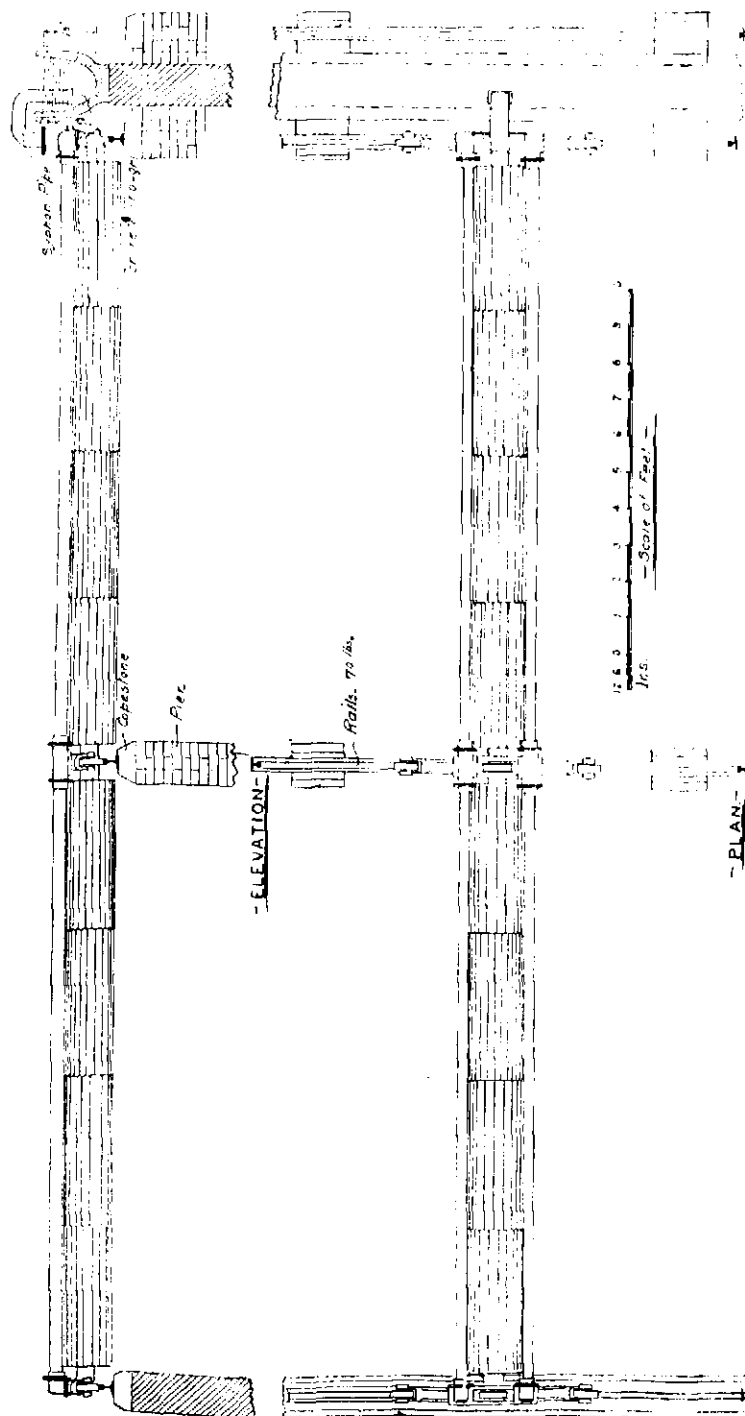
Figs. 698 and 699 show a new type of distributor which has lately come into general use and has given much satisfaction. It is Ham Baker's travelling distributor, made by Messrs Ham Baker & Co., Grosvenor Road, Westminster, S.W., who have kindly supplied me with the material from which the above drawings are made. This apparatus, which is simple in working, is on the principle of a water-wheel and runs on three rails, and is supplied from the effluent channel by a syphon pipe, which, when once started, continues to work without attention as long as there is any effluent to supply it. There is an ordinary regulating valve on the apparatus, and besides there is a patent automatic valve whereby the traveller, when it reaches the end of its travel, stops and reverses, and so on, continually travelling backwards and forwards. This effect is caused by turning the liquid to the opposite sides of the distributor.

Before leaving the important subject of sewage disposal it would be well to say a few words on a vital aspect of the question, namely, that of the sterilisation of the effluent. Of course much of the theory of this question belongs more to the chemist than the engineer, but it is the putting into practice of such theories that the engineer must be fully acquainted with, especially nowadays, when the pollution of rivers demands so much attention. The absolute sterilisation of an effluent is, however, a very debatable question. Then, again, some sewages would lend themselves to the process much more readily than others. First we have the "ozone" process, which is a patent, and is effected by electricity. The cost would of course vary with the price of power. At 1d. per kilowatt it would cost about 5d., and at 1½d., 7½d. per 10,000 gallons; on the other hand, the judicious use of a water-power plant should materially reduce the above charges.

A cheaper method is that by means of hypochlorite. The plant is also cheaper to install. Hypochlorite is produced as follows:—Through a solution of common salt an electrical current is passed, with the result that the sodium and chlorine which are the constituents of common salt are split up, the chlorine being liberated at the positive electrode and the sodium at the negative. The sodium and chlorine then recombine with one part of oxygen. The result is hypochlorite, in which it is the free chlorine and oxygen which have the sterilising effects, the oxygen being released during the process. To effect purification about 22 gallons of saturated solution of sodium chloride and 4.4 gallons of saturated solution of magnesium chloride are placed in a tank, to which is added 185 gallons of water, whence it flows by a regulating apparatus to cells at the rate of about 25 gallons per hour. A current of 15 amperes is then passed through the cells. The liquid so produced could be used afterwards for disinfecting. The process of sterilising an effluent would, however, be specially valuable in the case of septic tank liquor before filtration, because it usually gives off such bad smells, and the process would not materially hinder the filtration; in fact, it has been found that the cleansing of the filters has been made an easier matter by such a process.

Copper sulphate in the crystallised form has been found to answer well. The effluent flowing over it dissolves it. The process is, however, an expensive

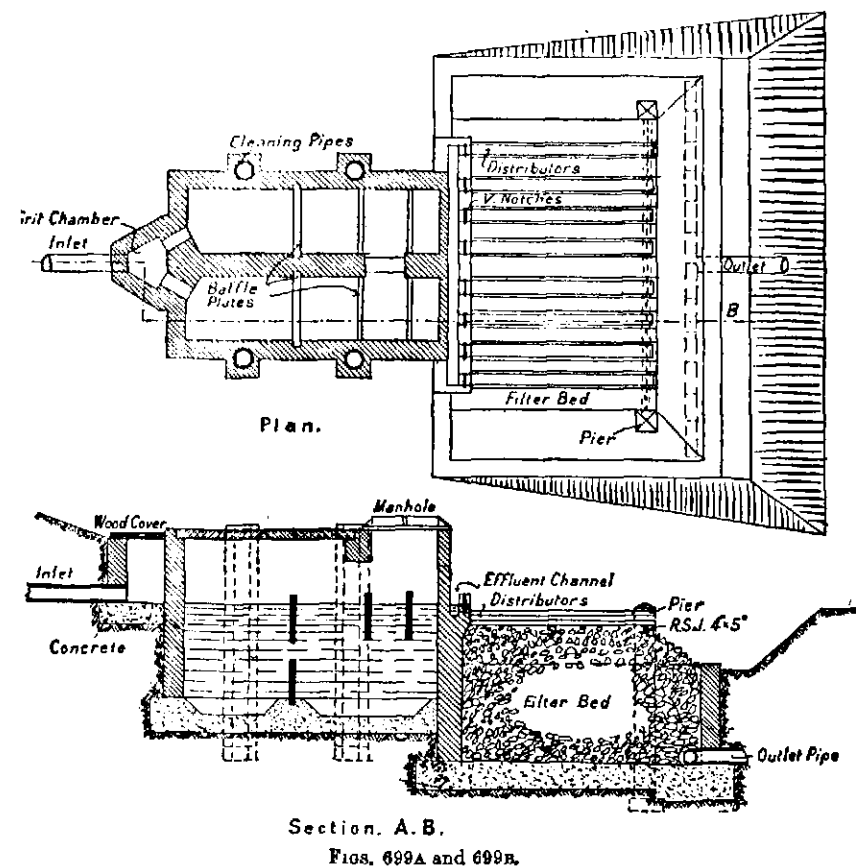




FIGS. 698 and 699

one. For a daily flow of 40,000 gallons, taking copper sulphate at 3d. per lb. and including labour, the cost would be about 5s. 6d. per day. A solution of bleaching powder in proportion of about 4 parts per 1,000,000 (of available chlorine) would successfully treat a filter effluent. Much valuable information on this subject is obtained in Bulletin No. 115 of the U.S.A. Department of Agriculture, issued in 1907.

In conclusion, we may say that sewage effluents could be economically treated by chemically prepared hypochlorite. It can also be prepared by an



Section. A. B.

FIGS. 699A and 699B.

electrolytic process, but economy would only result in works of large magnitude. The process certainly will reduce the smell nuisance, and is a valuable consideration in the case of large volumes of effluent going into tidal estuaries, or in places where fisheries or oyster-beds would be in any danger of pollution. Unfortunately, accurate data on the subject is scarce, in England at least, so that this is only a bare introduction to what, in the writer's opinion, might become a very useful addition to sewage disposal works if engineers, aided by expert chemists, took up the matter on practical lines.

In figs. 699A, 699B is illustrated for sake of completeness the system of the

to this mode of construction. For this class of work, however, a system known as the Simplex patent system would be eminently suited.

The essential principle of the system lies in the driving of a tube closed at the lower end, either by a loose point or a pair of hinged jaws, which open when the tube is being pulled up and permit concrete to be passed through, filling up the space left by the tube simultaneously with its withdrawal. The tube, or forme as it has been named, is of $\frac{3}{4}$ -in. or $\frac{1}{2}$ -in. lap-welded steel, generally 16 in. in diameter, and from 30 ft. to 40 ft. long, made up of two or three lengths, cross welded. The upper end is strengthened to withstand the impact of the hammer by a $\frac{3}{4}$ -in. riveted band 18 in. deep, and a narrower band $\frac{1}{2}$ in. thick is riveted on the bottom, where it bears on the shoe.

Fig. 699f shows a drawing of the ordinary form of Simplex piles.

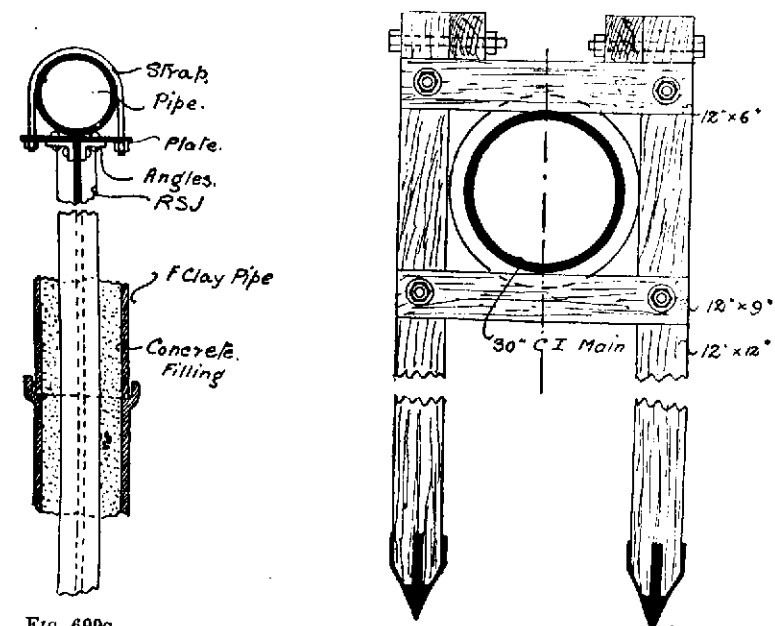


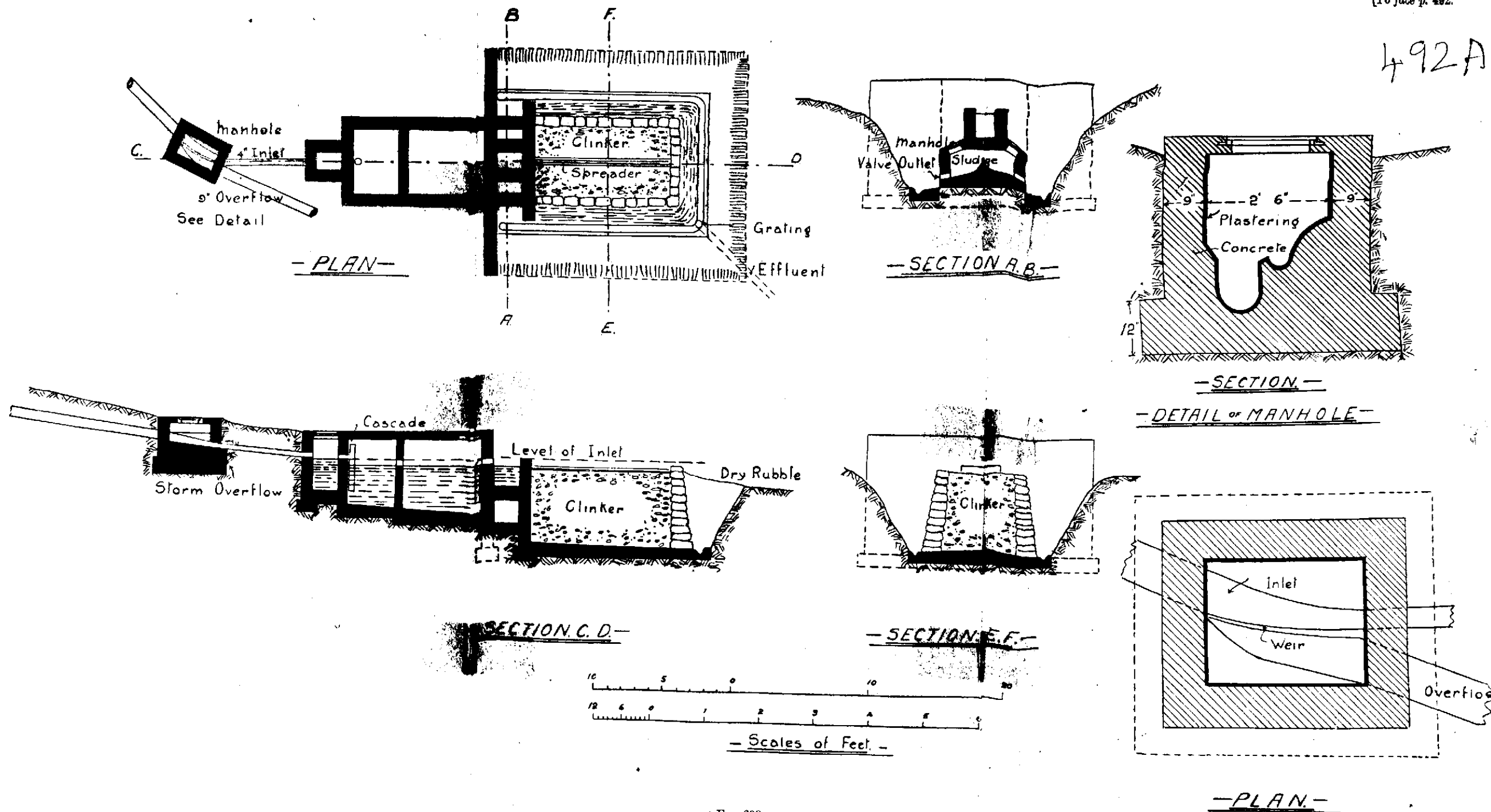
FIG. 699g.

FIG. 699h.

The "Simplex" pile is not necessarily reinforced, but where reinforcement is desirable it can easily be placed in these piles. Various types of reinforcement have been used, including rods wired together to form a cage, cylinders of expanded metal, or single bars, either round, square, or I-section. The reinforcement is usually placed in the tube before concreting, and if in the form of a cage, a special funnel is employed through which the concrete is poured, so as to fall down the middle of the cage without touching the horizontal wiring. The reinforcement of an ordinary concrete pile is necessary to enable the pile to be driven, but is of very little value in increasing the final carrying capacity of the pile. At the best, the difference in the coefficient of elasticity of the two materials only permits the steel to be calculated as carrying fourteen times as much as the same sectional area of concrete, while it costs from forty to fifty times as much. Clearly, therefore, the use

[To face p. 492.]

492A



* FIG. 699n.

Septic Tank Co. The tank and filter in question would be quite suitable for dealing with sewage from 40 people, or even 50 under favourable circumstances. The tank could be built of brick, made waterproof inside, or of concrete throughout. It is designed to meet the essential conditions previously set forth, and is air-tight, flagstone covers and a manhole being provided. Stone or slate would be used for the baffling plates, while a small grit chamber is also provided. Tanks of this kind, which the writer has had to deal with, have given more or less satisfaction. They are adapted to installations somewhat larger than the type of tank shown in figs. 735, 736, and smaller than that in fig. 689. They are frequently used with a Stoddart filter, which consists of a sheet of specially corrugated metal, with a number of notches cut out of the ridges, and small points attached to the under edge of the "valleys."

The distributor is fixed horizontally above the surface of the clinker. When the liquid flows on the distributor it overflows through the notches and runs down the under-surfaces of the "valleys," and is drawn by capillary attraction to the little points, from which it drops like rain on to the filter; it then passes down over the clinkers in a thin film, being in contact with the air the whole time, and taking air down with it.

The tank liquor is delivered into each trough over small V notches, which are frequently made of cast iron, but are preferably made of bronze, and very carefully set, so that each distributor gets an equal share of the tank liquor. This is a very important consideration, for upon the fact of this work being carried out with deal accuracy depends the whole success of filters of this description, and it is unfortunate to recall the fact that these filters have frequently given disappointing results, simply because this has not been properly carried out by experienced workmen. The older form of metal distributor has now been discarded in favour of those made from artificial stone or concrete, which could be well made of similar form to that shown in fig. 684. These, again, must be set true from end to end, and dead level. They rest on a small rolled steel joist, as shown in the drawings.

We have already dealt somewhat largely on the disposal of sewage in large inland towns, and the various methods which are employed to effect purification; it remains for us to make a few remarks in reference to small sea-board towns. It is the prevalent practice in such cases to let the sewage direct out to sea by means of specially constructed outfall sewers, often of the "tank" type already referred to. This is often the source of a large outlay of money, especially in tidal waters, chiefly because of the large amount of rise and fall, necessitating long outfalls, and also the fact that the work can only be done at low water, and is very often damaged by storms before completion. The proper point for the actual outfall is a few feet below low water of the ordinary spring tides. Then, again, the currents, as before mentioned, require very careful consideration. Where very high tides and a heavy rainfall are at all likely to be encountered at the same time, and the scheme is a gravitation one, some form of tank sewer is absolutely essential, because in a town which has a low-lying district contiguous to the sea, with steeply rising ground at the back, such conditions, unless amply provided for, will be sure to result in flooding of yards and basements, which, although it can be avoided by special traps, the main drainage should be so designed as to not necessitate these unsatisfactory appliances.

The actual outfall sewer is a piece of work which always gives rise to difficulty to the engineer, the work usually being very troublesome, and it is

very seldom that two cases occur alike; the shifting nature of the beach and other things will always crop up and alter circumstances, so that no hard and fast designs can be laid down. Cast-iron and steel pipes laid along the shore, with joints of the ball and socket type, have been tried with more or less success. This system makes the pipe practically flexible from end to end.* Such an expedient was resorted to about five years ago in the Waterford main

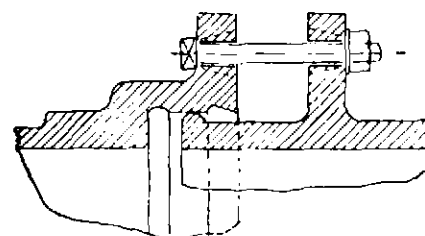


FIG. 699c.

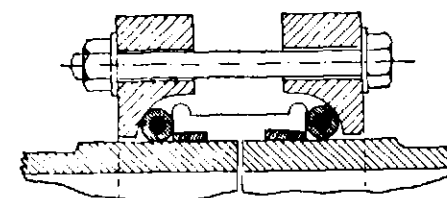


FIG. 699d.

drainage outfall into the river Suir. Of the flexible joint there are many types on the market, each claiming, of course, special advantages.

That made by the Stanton Ironworks is illustrated in fig. 699c, while Humphrey's patent flexible pipe coupling is shown in fig. 699d, and a very simple form made by Robert MacLaren & Co., Glasgow, is shown in fig. 699e. Perhaps the more usual practice is to fix the pipes to piles or concrete blocks,

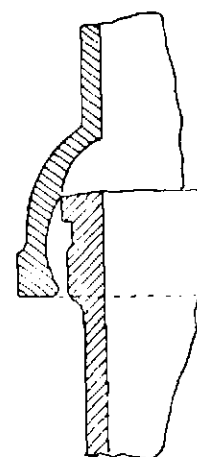


FIG. 699e.

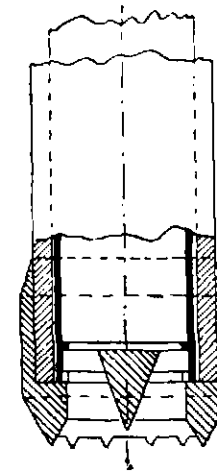


FIG. 699f.

or even building the outfall of masonry or concrete entirely. When piles are used, however, they require to be specially strong. The use of timber piles in tidal waters is far from satisfactory unless they are of greenheart, which is very expensive and seldom used. Steel has been tried with more or less success. No doubt, however, in the future, reinforced concrete will prove eminently adaptable to such cases. We have already in Chapter XX. referred

* *Proc. I.C.E. (Ireland)*, vol. xxxiii.

of steel is uneconomical where, as in most grounds, the piles are not subject to any stress but the direct load. The "Simplex" is the only concrete pile that can be placed without steel at all. Reinforcement against flexure under a central load is quite unnecessary in a pile which, having the support of the earth, cannot at all be regarded as a long column. The earth enables it to carry a load far in excess of what it could carry as a column, just as a needle if embedded in a cork can be driven through a penny—an experiment most of us have tried in our youthful days. If, however, there is any tendency for the ground to move and put a sheering stress on the piles, reinforcement becomes necessary, and the "Simplex" pile can be reinforced as heavily as seems desirable.

We also refer the reader to various forms of pile shown in Chapter XIV., while the system adopted by the engineer to the Durham main drainage for

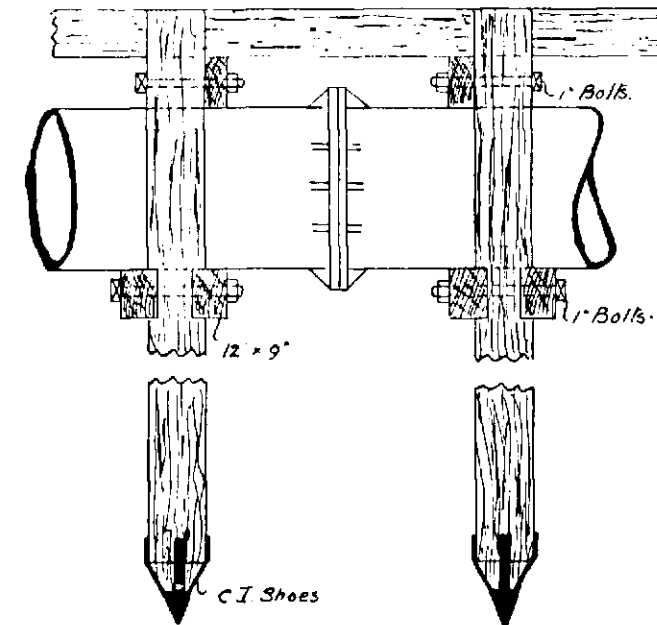


FIG. 699r.

carrying pipes along the riverside was to drive rolled steel joists to a solid foundation, then excavating the soil to well below low-water level, and finally encasing the pile in sanitary pipe plugged up with concrete, as shown by fig. 699a. Figs. 699u and 699v show timber piles carrying an outfall sewer of cast-iron flanged piping. It may be considered by some that steel pipes would be preferable to cast iron. For this class of work, however, it is not the case; rapid corrosion of steel is one drawback, while a good heavy cast-iron pipe adds considerably by its weight to the stability of the work. Cast-iron pipes could also be placed as shown in fig. 699x, but never fireclay pipes, because after-settlement would be sure to crack them. The writer knows only too well of a case of this kind with which he had recently to deal. It is advisable to terminate all tidal pipe lines in a tide flap shown, carefully set in concrete, in figs. 699t and 699m.

The plant shown in fig. 699x is a good example of a self contained sewage disposal works suitable for a small rural district or very large institution. The population it would satisfactorily serve would be about 120. The walls would be constructed in concrete, or brick and plastered inside in neat cement or otherwise made waterproof, the corners being preferably filled with cement

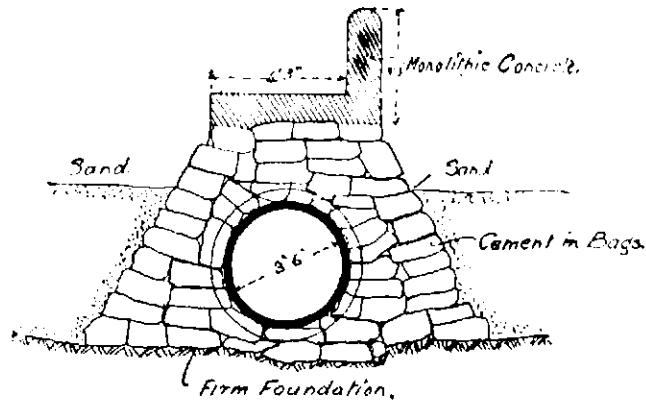


FIG. 699x.

and truly rounded afterwards. The drawings show the construction very clearly. The storm overflow is worthy of attention, and is constructed so that the depth of liquid in the small upper channel (see Enlarged Section), which

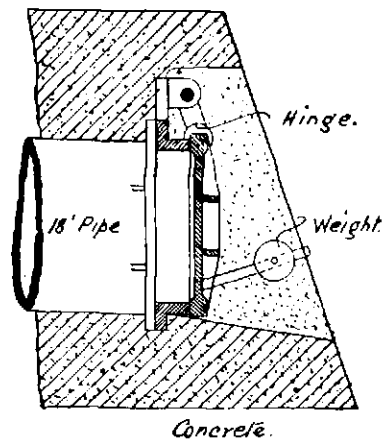


FIG. 699y. Section.

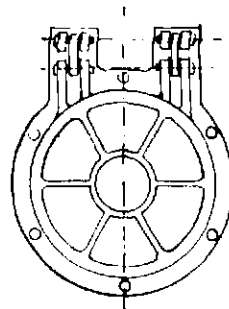


FIG. 699z. Elevation.

will just start to flow over the lip into the channel below, shall correspond to a flow of three times the dry-weather flow; anything above this will fall into the outlet channel and pass off to a system of land purification, in accordance with the present Local Government Board requirements. The design is well worth attention, because for small flows the parabolic leap weir fails to be efficient (some drainage engineers discountenance its use altogether). The

difficulty is to get the work done in concrete to sufficient exactness, together with sound construction, and to remedy this Messrs Adams & Co., the well-known sanitary manufacturers, have introduced a similar appliance made in cast iron, which is, in the writer's opinion, a splendid appliance. It is described as an adjustable storm-water diverter, by means of which the flow is divided in any desired proportion between the two outlets. Slotted bolt-holes allow of this division of flow being varied at will. The tank should fall towards the sludge chamber, and is provided with a concrete flat roof 6 in. thick, and is supported by an 18-in. x 6-in. pier from the division wall, which is formed where shown, having two 4-in. openings formed in the bottom for passage of sewage and sludge, the inlet from the grit chamber being formed of 4-in. pipes and bends, and secured to the wall with strong hooks (cranked), the outlet to the ponding chamber being formed as shown in 4-in. pipes in a similar way. Outlets to the sludge pit are formed at the bottom of the tank, and fitted with handstop sluice doors, with rod brackets and handle. The outlets from the sludge pits are similarly formed: 2-in. cast-iron ventilating pipes would be led from the sludge chamber in the body of the walls, terminating in bends and gratings over filter bed. There would be placed a suitable alternating gear in the ponding chamber. The filter is of the Stoddart pattern; it would not be suitable for small flows.

CHAPTER XXVIII.

ROYAL COMMISSION ON SEWAGE DISPOSAL.

An abstract from "Sewage Disposal," being the contents of the Report of the Royal Commission on Sewage Disposal 1908:—

Sewage is saline matter in solution, nitrogenous and carbonaceous matter in suspension + grit and mineral matter. Purification consists in removing the suspended matter, and oxidising the organic matter and ammonia. The latter is a biological process. The presence of corrosive sublimate would prevent such a biological process when pure water is mixed with sewage. Sewage absorbs dissolved oxygen, more or less rapidly, according to the degree of aeration of the water (*viz.* amount of dissolved oxygen). The organic matter first ferments, giving off carbon dioxide, water, and ammonia, and, secondly, the humus matter and ammonia ferments, producing carbon dioxide, nitric acid, and water.

The humus matters are waste products of the first stage, and are necessary to produce nitric acid, as without them nitrous acid alone is produced. As a consequence, the first stage becomes the carbon-oxidation stage, and the second the nitrogen-oxidation stage.

The sewage of individual towns varies in accordance as to whether there are many closets, how the storm water is treated, water supply, and manufacturers' waste. It is necessary to test each variety for oxygen absorbed by diluting the sewage, bottling up for two months or so, and then testing, or, a quicker way, by keeping strong sewage in a bottle which has known excess of air. It is the cellulose and fat which retard the process, because fat requires 6-1 parts of O by weight for complete oxidation. The strength of a sewage liquor is the amount of O by weight which 100,000 parts of the liquid would take up. Tanks should so be designed that the speed through them deposits the minerals, but yet allows the organic matter to go onwards; and where the tanks will not be cleaned out, often a grit chamber is essential, except in tanks dealing with less than 12,000 gallons per 24 hours. The term "sedimentation tank" denotes a tank where no chemicals are used. In these the sewage either flows slowly through or is drawn off by a floating arm. No rule as to time can be set out (Leeds and Sheffield allow 3 hours) because trades wastes prolong the process.

The tanks must be emptied after every first or second filling, because old sludge generates gas and carries particles with it; 10-15 hours would be required in tanks where the filters require to be kept free of suspended matter. There is little advantage in a secondary tank. The quiescent system requires more tank cleaning than the continuous. A septic tank does *not* digest all solid matter; the digestion or conversion into gas varies from 10-38 per cent.; and in such tanks the sewage is bacteriologically as

impure at exit as it is at entry, nor is such liquid more easily oxidised in filters. The sludge of a tank seldom cleaned contains about 10 per cent. less water than one often cleaned, and such sludge is the least offensive. The frequency of emptying depends on the particular works, coarse percolating beds being better able to deal with a "carry over" than contact beds. The frequency of working such beds is also important. Time should be allowed for the top sludge to dry and be scraped off. Generally small septic tanks do not want frequent cleaning, large ones do; but in this latter case only the best digested sludge should be taken out; one-third full is the limit in any case. No tanks should be entirely devoid of sludge. Burying sludge should be done in dry weather. The Dortmund form of tanks give easier settlement, but inferior sludge digestion to the rectangular form; very often sewage is in a septic condition on reaching the tanks. Based on the fact that 24 and 12 hours are the limits of septic tanks standing, the total tank capacity, exclusive of stand-by tanks, should be equal to the daily dry-weather flow of 24 hours. This is in consideration that the strength of the sewage is about 40 gallons per head per day. The rate in the tanks is best at or about 1 ft. per min., or even less. Such causes much activity by evolution of gas and increase of the free ammonia. Over-septicising causes an excess of H_2S and impaired oxidation in the filters. Closed tanks are best, as they reduce smell; sewage with much trades waste is less likely to smell. At Salford 75 per cent. of the suspended matter is removed in roughing filters. With chemicals the stay in or flow through tanks would be reduced to about 3 hours and 8 hours respectively. The principal chemical is alumina-ferrie, the cost varying from 24s. to 12s. per 1,000,000 gallons average daily flow, but alumina-ferrie and lime makes a better precipitant. The iron in a precipitant tends to colour the sewage.

Copperas requires a definite quantity of lime before addition. Lime alone is only useful in the case of acid sewage with A. F. + lime 8 grains, and 4 grains of each is the most economical quantity per gallon. Lime and copperas are cheaper, but require skilled supervision. The precipitant used in each case has to be suitable for the particular sewage.

For instance, lime alone is used in Birmingham, owing to the presence of much iron. Salts are of course an advantage, while sewage which contains much fat, such as the result of wool scouring, would require sulphuric acid, the fat being afterwards extracted from the sludge. The sulphuric acid neutralises the alkali. As regards adding the precipitant, in works where supervision is at hand, the liquid form is the best; at small works the solid, such as blocks of A. F., as it dissolves in proportion to the flow. Besides putting the blocks in the sewage, a good plan would be to place them in a box with a false bottom. In this is placed each day sufficient for 24 hours, and water allowed to drip on to it, so that during the day it practically all dissolves away, and at night there is not much left. The water, of course, is regulated by experiment.

Copperas is not used alone, but with lime, and is added after the lime. The sewage must be well mixed or baffled after the addition of the chemical. As regards the cost of various tanks per million gallons, we may state as follows:—

Continuous flow with chemicals	.	.	.	£3797
" " without "	.	.	.	5112
Quiescent " with "	.	.	.	5987
" " without "	.	.	.	5987
Open septic tanks	.	.	.	6891

The tanks in all cases must be large enough to deal with three times the dry-weather flow. Other tables may be found on R.C.S.D. p. 36; * average quantity is 10 grains per gallon. The average price is £2, 5s. per ton, which works out at £1, 8s. 8d. per 1,000,000 gallons. Average dry-weather sewage contains 35 in 100,000 parts of suspended matter, and would produce 1 ton 11 cwt. dry sludge per 1,000,000 gallons. The tank liquor would contain about 2.5 in 100,000 parts of such suspended matter after quiescent settlement, and 4.5 after continuous: if no chemicals were used, these figures become 7 and 10. Septic tanks would digest 30 per cent. of the sludge; this would give a 10 in 100,000 part (or .001 per cent.) tank liquor, and a remainder of sludge of 14.5 in 100,000 (or .00145 per cent.). Sludge is best disposed of by burial, septic tank sludge being specially hard to press, as it contains much grease. For such burial 1000 tons per year of wet (90 per cent. water) sludge can be dealt with on 1 acre of good soil. A chemical precipitation produces more sludge than a natural one, in the following order:—

	A	B
Quiescent process with chemicals	37.5	17
Continuous „ without „	35.5	16
Quiescent „ „ „	28.0	12
Continuous „ „ „	25.0	11
Septic tanks	14.5	6.5

Column A gives the parts per 100,000 left in tanks, column B gives tons per 1,000,000 gallons of dry-weather sewage of wet (90 per cent.) sludge. Then, taking a labourer at 21s. per week, and allowing the manager £130 per year, we have the following figures:—

	C	D	E
Quiescent with chemicals	8/4	11/5	£1 8 8
Continuous „ „	8/4	11/5	1 8 8
Quiescent without „	8/4	11/5	...
Continuous „ „	5/4	8/4	...
Septic tanks	5/4	8/4	...

Col. C = cost of sludge disposal on land.

Col. D = cost of tank supervision.

Each case allows one-third of the manager's salary.

Col. E = cost of chemicals per 1,000,000 gallons.

Col. F = cost of disposal for sludge (loan charges).

Col. G gives the total charges = C + D + E + F, all dealing with a million gallons of dry-weather sewage.

Col. H = strength of tank liquor.

Col. I = suspended matter in tank liquor in parts per 100,000.

* R.C.S.D. in the sequel refers to the Fifth Report of the Royal Commission on Sewage Disposal.

Col. J = rate of filtration per cubic yard per day.

Col. K = cubic yards of filtering material to filter 1 million gallons per day, all based on a dry weather flow.

Col. L = total cost of filtration (including loan charges).

Col. M = total cost of purification = G + L.

both for 1,000,000 gallons of dry-weather sewage.

F	G	H	I
£1 0 6	£3 8 11	50	1 to 4
0 13 9	3 2 2	60	3 to 6
0 19 11	1 19 8	70	5 to 8
0 17 3	1 10 11	80	10 to 15
1 2 2	1 14 6	80	10 to 15

J	K	L	M
175	5,714	£1 11 5	£5 0 4
150	6,666	1 16 3	4 18 5
100-125	8,928	2 1 11	4 1 7
75-100	11,494	2 12 8	4 3 7
75-100	11,494	2 12 8	4 7 2

The filters referred to are of coarse material 1"-3" in diameter. It will thus be seen that the total costs do not vary greatly. The most important consideration is how the sludge will be got rid of (B), to decide upon the method we adopt.

Domestic sewages are usually spoken of as very strong, average, and weak. The stronger sewage contains more suspended and colloidal matter than weak, and in a septic tank liquor this colloidal matter may be very considerable to clog a fine filter. Pumped sewage also varies, usually being strong to cut down plant expenses, and septic, owing to its waiting to be pumped. Strong sewage should be treated by chemicals, as it reduces smell both in sewage and sludge, and this sludge is also less costly to press. Weak sewage, being principally slop water, should be septically treated with average sewage. It is immaterial, except as regards local circumstances. For weak sewage, however, septic tanks are advantageous, except in cases where the sludge could not be buried. With brewery wastes, lime would have to be added for neutralisation; with alkali or soapy matter an acid is necessary, but when heavy matter is present, septic tanks are to be recommended, on account of the longer stay in the tanks.

Filtering Sewage.—The average water capacity of a primary contact bed may be taken as 33 per cent. of the original empty-tank capacity (viz. 66 per cent. of capacity in filtering material), and the secondary bed 40 per cent. ditto, or, in other words, 20-30 in 100,000 part sewage may be treated on contact beds at the rate of about 35 gallons per cub. yard of 3" material from 1-1½ fillings per day. 10-15 part sewage would be treated at 50 gallons per cub. yard for double contact on ½" to 1" material at 2 fillings per day. Again using fine material (¼"), a 1-4 in 100,000 part liquor for

double contact could be run at about 100-160 gallons per cub. yard per day, or 200-320 gallons if double contact. The medium would want washing about every 8 years. Percolating filters can deal with a stronger liquor up to 30 "parts" provided no fibrous matter is present. Thus average strength sewage could be filtered at about 50 gallons per cub. yard per day. The upper material would want renewal every 2 years, and the lower part washing. Tank liquors of 10-20 parts could be run at 100 gallons, and 4-7 parts at 100-150 gallons. In these three cases, however, the filtrant would want straining, and all such filters are liable to a top fungoid growth. This refers to 3" material. If the material was $\frac{1}{2}$ "-1" the above number of gallons must be halved. A $\frac{1}{4}$ " material would clog under such conditions, but a good tank liquor of 4-7 parts could run at 75-100 gallons per cub. yard per day, and a 1-4 part liquor 150-200 gallons. Sand filters could deal with a 4-7 part liquor at 50 gallons, and a 1-4 part liquor at 200-300 gallons; but such filters require to be used intermittently, and to be scraped about every 2 flushes or so, and to be flooded for filtration. In contact beds, it is generally supposed that the resting period is the most important for the bacteria, the ammonia being abstracted during the contact period and oxidised during rest. A matured bed is more effective than one not so. Contact beds should be built, and that water-tight and properly. The liquid should be drawn off quickly and quietly, the drains being as small as practicable. The valves should be placed convenient for taking samples, and feed channels made so as to take a weir notch for measuring. The limits of economical depth for beds are 2' 6" and 6' 0". Provide large material round the drains, and when taking a sample allow the first flush of usually bad effluent to run off. Contact beds are usually filled 3 times a day, having 2 hours' contact and 4 hours' rest; but new beds want a longer contact than old ones, and much diluted sewage of course requires less time, but in no case should the total time of contact exceed that of rest in 24 hours. The design should provide for 2 hours' contact. The periods of filling should be as far as possible from each other. Contact beds are liable to loss of capacity from—

(a) Consolidation, (b) Deposit, (c) Growth of organisms, (d) Suspended matter in the liquids. As regards (a) this is caused by weight and gradual breaking up of the filtering medium.

(b) This appears in a sort of jelly-like slime, which contains an enormous quantity of bacteria, absorbs O very rapidly, and gives off CO₂. It is one of the chief losses of capacity at the increase of efficiency. The fact of it absorbing O causes very free aeration. A 14 days' rest is sufficient or else this jelly will dry up, but such a rest means a gain of capacity. To keep a bed in a first-class condition it is necessary to prevent access of suspended or colloidal matter,—

1. Use materials not liable to disintegrate, and
2. Of even cubical shape.
3. Provide efficient drainage.
4. Give periodical rests, and
5. Fill and empty without disturbance.

No. 1 requires the most attention, and hence the practice of a fine top layer of sand which can easily be scraped. We can now conclude that chemical precipitation is the most efficient, but at the same time produces most sludge.

The filtering material may be economically washed and screened; the cost

of such, including making up of deficiencies, but not including any capital or maintenance charges on the machinery, which varies (table in R.C.S.D., p. 65) from 1s. to 2s. 6d. The washed material is usually of better value than unwashed. The ballast may be re-burned. This, including all labour, is done at 3s. 6d. per cub. yard. The usual material is coke and clinker; the coke, however, is liable to disintegration according to the size of the pieces. With crude 40 part sewage, the minimum would be 3" in clinker, for 20 parts $1\frac{1}{2}$ " do., and 10 parts about $\frac{1}{2}$ " do. The grading is unimportant, with the exception that a 6" layer of fine material on top and larger stuff round the sub-drains is essential to economy. As regards percolating filters, we may say the deeper the better according as levels permit, and the same quantity delivered per square yard. But when such is considered per cubic yard there is not much to choose, but the favour again lies with a deep filter on account of more perfect distribution of the four filtering materials, that is, Clinker, Coke, Slag, and Broken Brick; they are here arranged in order of purity of effluent (viz. clinker gives the best effluent), provided they are all of the same size. The same order also holds good as regards rate of filtration (viz. clinker is the slowest); also rough surfaces give better results than smooth. On the other hand, it would be an advantage to make medium size percolating filters shallow, in that there would be better aeration and a longer surface over which to distribute the suspended solids, but there should be perforated walls when fine material is used on top; these beds should be shallow, and the distribution should be made by a flushing syphon, so as to flood them. They must have frequent resting. We may finally conclude that as regards depth—

1. For fine or coarse materials the deeper the filter the better the effluent.
2. For coarse material in practice the same purification is obtained whether deep or shallow from the same cubic quantity of material.
3. But with fine material the filter should be shallow.
4. Fine or medium material produces the best effluents, but they clog with too much suspended matter, the limit being about a 50-100,000 part liquor.
5. Top growths and frost do not appear to deteriorate the workings except in coarse filters, where the former goes deep down; fine filters usually having such a growth scraped off.

The following are the essentials of good distribution:—

1. Uniform distribution over the whole surface.
2. District unaffected by wind and frost.
3. Capability of dealing with large and small flows.
4. Unchokable and easy to clean.
5. Working parts away from sewage liquor. As regards sprinklers in general,

- (a) The revolving type want much attention, and are liable to complete stoppage by wind and frost.
- (b) Spraying nozzles are cheap and efficient in cost and working, but want much attention and cause unequal distribution, and so are unsuitable for deep beds.
- (c) Flushing is only suitable for fine beds and comparatively weak sewage.
- (d) Tipping troughs can be used in small installation with the same precautions as type C, but the medium must not be too fine. Frost upsets them. All forms are likely more or less to raise a smell. The beds should be in a series of small "units," especially in types C and D. Any size would do for type B, but type A wants a circular bed. Where

the beds are coarse the bottoms tend to choke, so they must be made with a free base to allow of washing away the suspension; but in fine beds the top scraping does this also; free aeration round the sides of all beds is essential, so by resting a choked filter the suspension becomes granular and can be washed out. In the case of coarse beds such washing costs 1s. 6d. to 2s. 6d. per cub. yard. The top growth usually appears in early spring. Crude sewage seems to favour such growth more than settled sewage; there are also fungoid growths on the side of the channels, and it is particles of this which appear to choke the distributor tubes and necessitate cleaning every day. Caustic soda has the effect, when applied during rest in quantities of 1 litre to 1.6 sq. yards, of dissolving such growths which come down again, at first working in dark brown solids. Such a treatment could in all cases be done for 2½d. per square yard, and often much less, the solution of caustic soda being a 20 per cent. one. Generally speaking, such growths do not appear on fine beds. We will now compare contact beds and percolating filters, assuming that ample fall is available, and that the sewage has an average strength of 100, and the very best workmanship throughout. We now have to explain what a unit of purification is: Deduct the number of parts by weight of oxygen absorbed by 100,000 parts of effluent from the same weight by 100,000 parts of tank liquor, and multiply this by the number of gallons per cubic yard passing on to the filter in 24 hours.

The final conclusions come to regarding percolating filters are that—We can work coarse filters 6 feet deep at the rate of 100–125 gallons per cubic yard per day of strong (73–85 part*) septic tank liquor, or of half the strength at double the capacity, which gives a good effluent and about 9000 units of purification (R.C.S.D., p. 45). The above figures may also be used with settled sewage and precipitation liquor. We may look for an average of 3000–4000 degrees of purification on double contact beds with weak liquors, the same on single contact beds, while percolating filters will give 2500–3500 in crude or partially settled sewage, and septic tank liquor on double beds will give 3500–4500, and on percolating beds 6'–9' deep 7000–11,000. The amount of liquor per cub. yard varies in direct inverse to the strength. In the above figures in reference to settled or crude sewage we can take 60 gallons of strength 60 per cub. yard, and with septic liquor on D.C. beds 70 of 70, and on percolating filters 96 of 80, which all goes to say in the case of 70 of 70 we could treat 140 of 35, etc., etc.

(e) *Land Treatment*.—It has been found that such treatment without elimination of the suspended solids will give 98 per cent. purification as compared with contact beds 93.4 per cent., and percolating filters 99 per cent., the two latter referring to sewage without suspended solids, but in land treatment there is always risk of sewage passing away unpurified with the best treatment. We can put the sewage of 1000 persons on one acre of good land, but such land as clay would only treat that from 100 persons or so, as per table.

Surface irrigation requires more land. Differing from filters, it would be better on land (which may always be considered as fine material) to treat one volume of strong sewage than two volumes of half the strength, allowing each person 30 gallons per day, because in the latter case water-logging is very liable. Sewage thus treated on good land is not usually

* Of suspended matter in 100,000.

	A	B	C	D	E	F	G	H	K	L	Single contact in this case.
Quiescent settlement + chemicals	£8,527	£7,376	1.26	0.73	8/4	5/4½	7/9	£3 8 11	£2 3 11	£5 12 10	
Continuous do. + do.	14,898	8,836	2.03	0.91	8/4	5/4½	9/3	3 2 2	3 4 7	6 6 9	
Quiescent do. - do.	22,080	10,596	3.21	1.095	8/4	5/4½	14/7	1 19 8	4 11 8	6 11 4	
Continuous do. - do.	26,846	13,824	4.18	1.16	8/4	5/4½	19.9	1 10 11	5 11 4	7 2 3	
Septic tank liquor	26,846	13,824	4.18	1.46	8/4	5/4½	19.9	1 14 6	5 11 4	7 5 6	

Col. A = Initial cost of filters, including purchase of filtering material for 1,000,000 gallons per day contact beds.

" B = do. do. for percolating filters.

" C = Acres of land required for such filters, as A } cost about £100 per acre.

" D = do. do. as B }

" E = Labour and supervision (labourer at 21s. and manager £43 proportion) for contact bed per 1,000,000 gallons.

" F = do. do. percolating filters.

" G = Cost of washing out and renewal to beds (contact beds) per 1,000,000 gallons.

" H = Total preliminary treatment (same as G).

" K = Total cost of filtration (= E + G + loan charges) for contact beds.

" L = Total cost of purification with contact beds, by which we see that contact beds are the most expensive.

	A	B	
Gravel	1	1	A referring to broad irrigation and B to intermittent filtration. Loam is clay and sand in mixture, clay making it heavy.
Light Loam	1½	1	
Heavy Loam	2½	2	
Chalk	unsuitable	2½	
Peat	3½	uncl.	
Clay			

settled, but should be so done on clay, etc.; in fact, it is advisable in all cases, to prevent risk of nuisance. Also the amount of effluent is less, owing to evaporation and absorption by the crops. The manurial value of sewage depends on the urea of the urine, the fermenting of which causes ammonia, but there are also organic nitrogen, with phosphates and salts of potash. Shallow wells in the vicinity of sewage farms are (unless certified to the contrary) dangerous; deep wells sunk in impervious strata fairly safe. Chalk wells are very dangerous; in fact, sewage farms on chalk at all should be considered carefully; the reason is that chalk does not purify sewage, neither is peat suitable for sewage farms.

Sludge Disposal.—Sludge is the “bugbear” of sewage disposal. It contains small quantities of nitrogenous compounds and phosphates and grease, but the bulk is mineral matter together with cellulose. The sewage of Kingston-on-Thames is made into what is called native guano, the average composition of which is—

Moisture	25 per cent.
Matter (volatile origin)	37 ”
Matter (non-volatile)	37 ”

This latter is principally grit, 22 per cent.

Oxides of iron and alum	10 per cent.
Magnesia	3 ”

The sewage is treated on the A.B.C. process; globe fertiliser is prepared on a similar system. It contains less water and less grit in the non-volatile matter. It is produced at 10s. per ton, which includes 4s. 6d. for pressing wet sludge; much dust (injurious to workmen) is produced during manufacture. Sludge is often put out to sea. The total cost of such disposal is in London about 4·6d. per ton, and in Glasgow 6·2d. per ton, and in Dublin just over 5d. per ton.

Sludge pressing is carried on at various places. The process removes about 35 per cent. of the water, about 1 per cent. of milk of lime being added, more being required if there is much grease in the sewage. Pressed sludge will not smell if kept dry, but it will if it gets wet, so it is usual to store under cover. The composition, excluding water, is about 25 per cent. of grit and 10 per cent. of lime. Sludge pressing costs from 7s. 3d. to 8s. 6d. per ton of wet (90 per cent.) sludge. Sludge may be disposed of by burying. Small V-shape trenches are dug and allowed a month or two to dry, the sludge is run in, and the excavation tipped in on top. The area is then left to dry, ploughed over, and used again. Smell is likely to arise. The area of land required of average quality is ½ acre for every 1000 tons of wet sludge per annum. Stiff

clay land would be unsuitable. Snow and frost interfere with the process. The cost of such disposal may be set down as 6d. per ton of wet sludge, which includes sludge pumping. Lagooning is a method of sludge disposal by spreading it out over a shallow lagoon, which has a clinker bottom, under-drained. The sludge, when dried, which takes from 2 to 6 months, is dug up and buried. The system causes smell. Septic tank sludge, which is usually free from smell, may simply be put on land 3”–4” deep, where it dries, cracks, and yields rich black soil. The cost of such a procedure is just under 1½d. per ton of wet sludge. 90 per cent. sludge cake can be burned with house refuse in a proper furnace. The resulting clinker makes good mortar. The process costs about 5s. 2d. per ton of pressed cake. The removal of grease from sewage which contains a large quantity of such is done as follows: H_2SO_4 is added in quantities of 10 in 100,000 parts, after passing the detritus pits. It is then tank-treated, and the effluent let off in the usual way. The sludge is then subjected to heat (212° F.) by means of steam, and more H_2SO_4 in cast-iron vats, and heavily pressed while hot. The fat is then abstracted and prepared for sale by boiling with fat acid and black oxide of manganese. Yet another system is sludge distillation. The sludge is pressed and dried to contain about 20 per cent. of water; when its bulk and weight is reduced about 50 per cent. this material is put into a retort something similar to a gas producer, and the gases drawn off, the oil, tar, and ammonia are abstracted, and the gas burnt to heat the producer, which remains at a temperature of 1800° F. Steam is blown into the retort. The residue, a fine powder, can be applied as direct manure, or it will make cement. The average yield of residue is 5 per cent. of ammonia sulphate = 100 lbs. per ton of dry (20 per cent.) sewage cake. As regards the actual manurial value of sewage sludge, when used in quantities sufficient to apply 40 lbs. of nitrogen (amm. sulph.), the yield of corn and straw will increase about 10 per cent. The best sludges are those which contain most water and lime, and the manufactured cakes are better than crude sludge; but owing to the large percentage of grit its value is counteracted by long-distance transport. Mill dams are always liable to get choked with sludges; this requires periodic removal, and is usually in a putrid state. If at all practicable, such should not be pushed further down the stream, but be buried or disposed of on land as above.

“*Trade Effluents.*”—Trade effluents in all cases increase the difficulties of sewage purification. Brewery waste necessitates much lime; the sludge is very likely to cause nuisance by the evolution of much H_2S . Wool-scouring liquor, as before stated, is subject to “cracking” by H_2SO_4 . Waste liquors from galvanising works send down much iron, but if enough lime is added it is an advantage in purification; woolly fibre gives rise to much difficulty, as the filters will clog. Tannery waste demands a longer treatment and more frequent cleansing of the settling-tanks. Both septic tanks and percolating filters will cause more nuisance than other systems. Strong domestic sewage is also so liable, but strong sewage + manufacturers’ waste not so. A chlorine solution is an advantage to this difficulty. Land treatment of sewage in general is more advantageous than artificial ways, provided we can purchase the land at or about £100 per acre, and that the land is suitable to such a process. As the primary treatments cost about the same in all processes, the selection of such is a matter of local circumstances and cost of sludge disposal.

“*Storm Water.*”—On the combined system, a fairly heavy rainfall will increase the dry-weather flow 50 per cent. For the first hour or so after

a storm we have an increase at the outfall of organic matter in solution and suspended solids; in fact, a storm usually causes a big rush of very foul sewage, which on no account should go over storm overflow; indeed, in hilly places the flow may suddenly rise to 30 times dry-weather. The size of such a tank should be 6 hours' dry-weather flow. Storm-water filters are of practically no advantage, because not being in regular use they are liable to choke. The L.G.B. regulations are that three times the dry-weather flow is to be treated in the ordinary way, and three volumes on storm-water filters. These, as suggested above, should be replaced by stand-by tanks, and treatment on the ordinary filters resorted to. In any case, surface water may be looked upon as putrid. Unpurified sewage passing into rivers will have the following deleterious effects:—

1. De-aeration, and consequent injury to fish.
2. Smell.
3. Fungoid growths.
4. Accumulation of sludge behind weirs.
5. Presence of intestinal organisms; and the latter condition is quite possible even from clarified effluents.

6. The formation of black putrid mud in backwaters. In order to test an effluent we must find out the amount of suspended matter it contains; and after the removal of such, the amount of oxygen it takes up from water.

A good effluent should not contain more than 3 in 100,000 of suspended matter; and after being filtered, the filter-paper should not absorb more than—

(a) .5 in 100,000 by weight of dissolved or atmospheric oxygen in 24 hours.

(b) 1 in 100,000 of d. or a. O in 48 hours.

(c) 1.5 in 100,000 of d. or a. O in 5 days.

Effluents in streams are always likely to encourage growth, due to the presence of nitrogen. It is the grey growths which are liable to raise nuisance when they decompose, green growths being practically harmless. The following conclusions in reference to effluents (apart from suspended solids) may be of interest, viz. :—

Effluent which takes up very little dissolved O from water rarely fails in the incubation test, and conversely; and an effluent to be harmless when let off into a stream must not take up dissolved O from the water at a greater rate than that water can re-aerate itself from the atmosphere. Many effluents, particularly those from contact beds, are completely (more or less) de-aerated.

CHAPTER XXIX.

COPY OF THE ENGINEER'S REPORT TO THE LOCAL GOVERNMENT BOARD ON THE SALFORD SEWAGE WORKS.

The following chapter consists of a verbatim Report by Mr J. Corbett, made to the Local Government Board, on the subject of the Sewage Works of the County Borough of Salford.

The Salford Sewage Works, in their completed state, including a few items not yet carried out, but intended to be finally completed within a few months, may be described as follows. The process is threefold.

1. Chemical precipitation by continuous flow in tanks.
2. Straining through gravel roughing filters.
3. Bacterial purification by aerated filters or beds, supplied through spray-jets.

About one-fourth the total flow of sewage comes in by gravitation, and three-fourths is pumped from the main intercepting sewer, which is laid near the river in its winding course through the borough, crossing below the river at three places, without requiring syphons, and serving to drain the land water.

The dry-weather or minimum daily flow of sewage from the borough population of nearly 230,000 is about 8,000,000 gallons, of which about 5,500,000 gallons flow between 9 a.m. and 9 p.m., and 2,500,000 gallons between 9 p.m. and 9 a.m.

The average daily flow is 12 to 13,000,000 gallons, and the greatest flow provided for is 32,000,000 gallons, or fourfold the dry-weather flow.

Of the 8,000,000 dry-weather flow, about 4,600,000 gallons is domestic sewage (the estimated quantity of the water supply), about 2,000,000 is liquid trade refuse, and the remaining 1,400,000 gallons is from subsoil water, this item increasing considerably in wet weather.

The ultimate flow provided for, 32,000,000 gallons, is 139 gallons per head of the present population.

The main intercepting sewer has a sump sunk 2 feet below its invert, from which a chain bucket dredger daily lifts any deposit of silt, by steam power, into tram trucks, which convey it to the sludge tanks, and so out to sea.

There are three screening chambers, controlled by valves, each with an iron grating 6 ft. wide and 13 ft. high, with bars $\frac{1}{4}$ in. apart. The usual raking apparatus is provided, worked by steam power.

The sewage then passes through a 45-in. brick culvert to the pumping engines.

Each of the three pumping engines has a 30-in. suction and delivery pipes, separate throughout.

The two original pumping engines are alike, each being a vertical compound engine, with $24\frac{1}{2}$ -in. H.P. and $40\frac{3}{8}$ -in. L.P. cylinders, connected to cranks at 90° , and with a double-acting pump at some distance below each steam cylinder, 31 in. diameter, all of 6 ft. stroke, and up to 17 revolutions per minute.

(Originally made by Messrs J. Watts & Co., Soho Works, and greatly improved by local firms.)

In 1906 a centrifugal pump and vertical compound tandem engine was added, made by Messrs Tangyes, Ltd., with cylinders 17 in. and 25 in., by 12-in. stroke, and up to 194 revolutions per minute; the fan 60 in. diameter. Inlet and outlet 23 in. diameter.

Each of these three pumps, when in good condition, can raise 13,000,000 gallons of sewage per 24 hours, the gross lift being from 20 to 30 ft. So two engines can deal with four times the dry-weather flow, leaving one engine in reserve.

There are four steam boilers, each 28 ft. by 7 ft., of the Lancashire type, with Galloway tubes, worked at 64 lbs. pressure. Their furnaces are fitted with steam-jet forced draught, to facilitate the use of inferior fuel.

It is proposed to provide boilers of higher pressure, worked by refuse destructors, so as to save the cost of fuel.

The sewage pump is delivered into a mixing chamber at the east end of the tanks; about 10 ft. cube, with a mixing screw 6 ft. diameter on a vertical shaft. Here is connected the gravitation flow; and the milk of lime precipitant is added.

There are sluice valves to provide for a by-pass flow (during repairs, etc.) of all the sewage along the gravitation channel to the west end of the tanks; but the usual flow is through two silt pits, one about 100 ft., the other 50 ft. long, each 10 ft. wide and 14 ft. deep, each with an 18-in. valve and silt discharge pipe to the sludge tanks.

Between the two silt pits is a weir, where the flow of sewage is gauged by an automatic recorder; and below this weir the dissolved salts of iron precipitant is added. (The preparation of the precipitants is described later.)

The sewage then enters the central channel, 10 ft. wide, about 4 ft. deep, and 550 ft. long, from which it is passed into the tanks on each side by large sluices. There is a system of large sluices between the several tanks, also outlet sluices to the tank effluent channels, one on each side of the range of tanks, so that the flow can be run through all the ten tanks in series; or through groups of five, three, or two tanks; or through single tanks in parallel, any tank being shut off, emptied, and cleansed when required.

The ten precipitation tanks have a total capacity of about 5,000,000 gallons, each tank being about 115 ft. by 82 ft., and from 7 to 10 ft. deep. (The disposal of sewage sludge from these tanks is described later.)

The tank effluent channels lead to a cross channel at the east end of the tanks, in which are 20-in. valves for supplying each of the roughing filters, six in all, which adjoin this channel. Each roughing filter has a row of inlet holes along its west end, supplied from a chamber in the wall below the cross channel. The floor is of perforated tiles on short legs, thus forming an outlet channel leading to an outlet pipe 20 in. diameter, controlled by a valve in the valve chamber; and from this chamber three 30-in. pipes convey the effluent to the aerating filters or bacteria beds.

The roughing filter valves are arranged so as to provide for reversing the

current in any one of the six filters, and thus "upward washing" it, the washing passing to the sludge tanks or to the sewer, as preferred.

Also, for cleansing these roughing filters, each of the six is divided by screen walls into three bays, thus forming together eighteen bays, which can be "upward washed" one by one. Each bay of about 105 square yards area has a system of air-blowing pipes fixed near its floor, through which air is blown at a pressure of 5 lbs. per sq. in., through 4800 holes of $\frac{1}{8}$ -in. bore, thereby disturbing the gravel and speeding the upward flow of the washing water.

These roughing filters are made of 3 ft. depth of fine gravel between $\frac{3}{16}$ and $\frac{5}{16}$ diameter. They have to be "upward washed" and blown through once a day, or oftener when the flow of sewage is great. Their purpose is to remove matter in suspension and colloidal matter.

The three 30-in. pipes above mentioned are along the north or upper end of the bacteria beds, with a 15-in. branch and control valve to each of the 15 beds first constructed, and with an 18-in. branch and valve to each pair of the beds subsequently formed. The 30-in. pipes have also valves at their inlets, so as to control the beds in three groups.

When the first 15 beds were designed in 1894 there was but little experience as to the durability of bacteria beds, so they were arranged with a strong tram rail supported on cast-iron pillars at each 31-ft. width, each alternate line of pillars also carrying a light tram rail at 2-ft. gauge from the strong tram rail, so that tram trucks of 2-ft. gauge might thereby bring the filtering material to its place. A light portable bridge with a turntable then carried the trucks across the width of a bed, so that the material could be dropped exactly where it was wanted.

The strong rails were intended to carry a machine something like a steam-power travelling crane, by which the surface of the beds could be raked, or the material refilled into tram trucks and carried away for cleansing or renewal when required.

The supply of clarified sewage to each bed passes through a 15-in. pipe at the base of the cast-iron pillars, and each pillar forms a rising branch up to a horizontal pipe 4-in. diameter, fixed above the surface of the bed, at 10-ft. centres, with six spray-jets across the 31-ft. breadth of beds, each spray-jet serving an area 10 ft. by 5 ft. 2 in.

In the later constructed beds the tram rails are omitted, and the supply pipes are carried above the surface on brick pillars, with 3-in. branches right and left, so as to cover a double bed of 62-ft. breadth. All the distributing pipes have emptying valves and pipes leading to a sewer, to avoid risk of frost when resting.

The filter floors are of concrete, with culverts at 32-ft. spaces for conveying the filtrate to the 4-ft. final outfall culvert at the south end of the beds. The concrete floors are covered with perforated tiles on legs 4 in. long, so as to provide ample ventilation below the beds. The earlier beds have ventilating manholes at each end of each culvert, and the later beds have vent shafts through the brick piers carrying their supply pipes; they have also open sides allowing free air-way to or from their raised tile floors, but the earlier beds are enclosed with walls. There are no dividing walls between the adjoining beds; they form really one continuous bulk.

The filtering material is crushed cinders and clinkers, from about $\frac{3}{16}$ to $\frac{1}{2}$ in. in the earlier beds, and from $\frac{1}{4}$ to $1\frac{1}{4}$ in the later beds, with some coarser

material at the bottom. Thirteen of the earlier beds are 5 ft. deep, two are 8 ft., and the newer ones are about 7 ft. deep.

Owing to these beds having been worked for several years before the roughing filters were completed and provided with cleansing apparatus, there is much more clogging of the beds than was expected; but after they have been once thoroughly cleansed, it is expected that they will require but little renewal or cleansing in future years. Their surface is raked by hand labour each few days.

The spray-jets, when working under the full available head of about 8 ft., deliver at the rate of 50 gallons per sq. yd. per hour, or 1200 gallons per 24 hours if run continuously. There are 39,000 sq. yds. of beds, so each hour's flow of all the spray-jets will discharge 1,950,000, say 2,000,000 gallons. So, with the minimum or dry-weather flow of 8,000,000 gallons per day, each bay must be worked about 4 hours and rested 20 hours; and with the maximum flow provided for, fourfold the minimum flow, or 32,000,000 gallons per day, each bay must be worked 16 hours and rested 8 hours; but the working hours can be extended if desired by checking the flow by the control valves.

It has been found best to run each bed for two hours and then rest it for ten hours or less, according to the quantity to be dealt with.

Thus the minimum daily flow per sq. yd. of bed is about 205 gallons, the average about 330 gallons, and the maximum 820 gallons; but this large flow will only occur when the sewage is greatly diluted with rain water.

As the beds average 2 yds. in depth, these flows are only, per cub. yd. of medium, 103 gallons per day minimum flow, 165 gallons average; and 410 gallons maximum flow when diluted with rain water.

The filtrate from these beds flows into the old culvert which formerly conveyed the intercepted sewage of the borough into the river Irwell before the sewage works were constructed; but the Irwell is now transformed into the Manchester Ship Canal.

Turning now to the disposal of the sewage sludge from the precipitation tanks, the floor of each tank slopes to an outlet pipe controlled by a valve in a subway below the central channel, and discharging into a channel along the said subway, and thence through an 18-in. pipe and a further length of open channel to the sludge tanks.

Men with push-boards sweep the sludge across the tank floor to the outlet, wading in it with waterproof thigh-boots.

The outlet pipe in each tank has above it two 2-ft. lengths of cast-iron pipe, with valve seatings, so that the outlet is first used to discharge the supernatant water of the tank down to 4 ft. above the outlet level.

Then one length of pipe can be raised and the water run down 2 ft. above the outlet level; and the flow along the subway channel is all this time turned into the main sewer. But when the second pipe is raised, and the sewage sludge deposited on the tank floor begins to flow, the channel is connected to the sludge tanks.

There are two sludge tanks, circular and saucer-shaped, 100 ft. diameter and 9 ft. deep in the middle, the two holding 3000 tons of sludge. There is a supply channel around each sludge tank, with holes through the wall to distribute the sludge. There is also a sludge supply trough extending to near the centre of each tank. From the centre of the tank bottom an 18-in. suction pipe leads to the sludge pumps; and also at the centre is a 6-in. skimming pipe leading to the sewer, and fitted with a "telescope-pipe" and

supporting float to skim water from near the surface, whether the tank be full or not. Thus the densified sludge sinks to the bottom, and the separated liquid is passed to the sewer, and so again through the tanks, etc.

Between the two sludge tanks is the sludge pump-house, with two sludge pumps of "Tangye's Special Pump" type, each with steam cylinder 16-in. diameter, double-acting piston pump 12-in. diameter, both 24-in. stroke, running up to about 35 double strokes per minute, each pump delivering about 200 tons of sludge per hour. These pumps work very satisfactorily, even when dealing with sludge containing 20 or 30 per cent. of solid matter.

From these pumps an 18-in. pipe, 200 yds. long, conveys the sludge to the sludge steamer *Salford*, at a wharf on the Ship Canal.

This steamer is 170 ft. long, 32 ft. beam, about 11 ft. draught loaded, and carries 600 tons of sludge in four tanks formed above the light-load line so as to discharge through eight 18-in. valves and pipes through the bottom of the ship.

The distance to beyond the North-west Lightship off Liverpool is 64 miles, or 128 miles the round trip; and the steamer usually makes four or five trips a week, going with each alternate tide. Her fastest round trip was made in about 15 hours.

The preparation of the precipitants, lime and iron salts, used for the sewage, is provided for in the lime-house.

A branch from the L. & N.-W. R. Co.'s dock line brings the lime trucks directly from the Hoffman kilns at the Buxton quarries into the lime-house, where the lime is stored for use. A workman fills a wheelbarrow with lime, passes with it up a lift to a raised floor, tips it into one of the lime-slaking pans, and there it is dissolved. The apparatus is in duplicate to provide for repairs, etc.; each set includes two slaking pans with revolving mixers and mixing pan.

When the pan of lime is slaked more water is added, and it flows through a grid, where stones, etc., are intercepted, and so into the mixing pan, which is like a mortar mill, with two heavy rollers revolving in it, by which any unslaked pellets of lime are crushed, and thence it flows to the mixing chamber already described.

The salts of iron used are a by-product from a local chemical works, very similar to "copperas." This is easily dissolved in cold water in two mixing pans, but steam pipes are also provided for use if required.

The water used is the tank effluent, raised by small centrifugal pumps. A duplicate couple of steam engines work the machinery.

The chemical laboratory adjoins the lime-house; here the resident chemist makes daily analyses of samples taken hourly, and mixed, to show each day's average of the crude sewage, the tank effluent, the roughing filter effluent, and the final filtrates from the different bacteria beds. The sewage sludge is also sampled and analysed.

There are also on the works a smiths' and mechanics' shop, ample stores, offices, men's rooms, weighing machines, etc., also the manager's house, with a large committee-room.

The site of the works is 30½ acres (some land having been sold for the Ship Canal), and of this about 9½ acres is still unused, and 2 acres is only used for gardens, etc., the actual sewage works thus occupying only 19½ acres.

CHAPTER XXX.

SANITATION AND HOUSE DRAINAGE.

The Ventilation and Warming of Buildings.

ONE of the commonest constituents of the atmosphere is carbon dioxide. Like all other gases, it is subject to physical changes of state as the result of increased pressure and diminishing temperature, and can be made to undergo liquefaction.

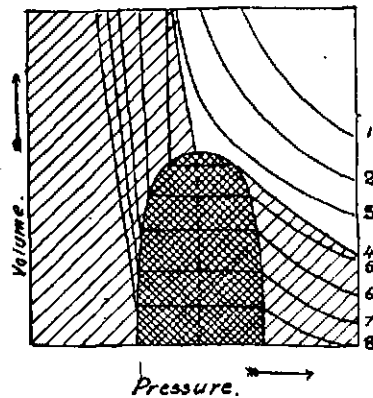


FIG. 701.

Fig. 701 represents diagrammatically the changes it undergoes when subjected to increased pressure. At a temperature of 50° C. a fairly uniform curve is produced (No. 1), but No. 3, plotted when the temperature has fallen to 30° C., shows the tendency of the gas to liquefy. This temperature is called the critical temperature for carbon dioxide; it is the highest temperature at which increased pressure produces liquefaction. In other words, carbon dioxide cannot be converted into a liquid by pressure until it has been cooled down to this critical pressure of 30° C. In the diagram the white area shows the gaseous and the shaded area the liquid states. The cross-hatched portion

shows the period of liquefaction (fig. 701).

Atmosphere and its Pollution.—The atmosphere is composed approximately of

78 per cent. of nitrogen	} by volume.
21 " " oxygen	
1 " " argon, etc.	

Argon is a peculiar gas. It is about $1\frac{1}{2}$ times heavier than atmospheric air, and will not combine with any other substance.

The atmosphere also contains traces of carbon dioxide, ammonia, organic matter, ozone, and watery vapour. Ozone is a form of oxygen only present in pure atmosphere. It is written as O_3 .

It is found that the carbon dioxide (CO_2) present in air, as a result of expiration, corresponds very closely to the organic matter from the same source, and hence the reason that we take the amount of CO_2 present in any atmosphere as a standard of purity. Now when CO_2 is present in as much as 2 parts in 1000 it is apparent to the senses; that is to say, added CO_2

in addition to the 3 to 4 parts in 1000 usually present. Therefore if we know the average quantity of CO_2 which a person gives off per hour, which we generally consider to be equal to 6 cub. ft. per hour in repose, we find that each individual will require 3000 cub. ft. of air-space per hour; and as the air in a room cannot be changed more than three times per hour with comfort, in cold weather, 1000 cub. ft. space should be the minimum allowance for each person. As examples, however, we have barracks—allow 600 cub. ft.

The laws relating to common lodging-houses only require 300 ft.; dairies should provide for 800 cub. ft. per cow; and in estimating these quantities 13 ft. may be taken as the maximum useful height of a room.

The chief gaseous impurities in air are—

CO , CO_2 , Coal gas, H_2S ,

nitrous acid, suspended matter, dust, seeds of plants, spores of fungi and bacteria.

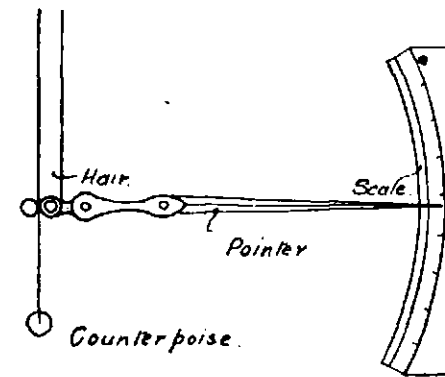


FIG. 702.

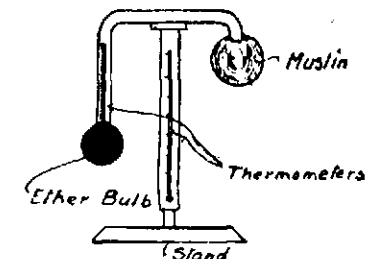


FIG. 703.

Now the artificial light in rooms will tend to foul the atmosphere to a certain extent, except, of course, electric light. For instance, when 1 cubic foot of coal gas is burnt we evolve 2 cub. ft. of CO_2 , while 1 pound of oil burnt will give off 23 cub. ft. of CO_2 . It can be proved that in 4% of CO_2 (it is, of course, a question of the amount of O present; CO_2 is merely a diluent, and not a poison) a candle will cease to burn, and that the air is dangerous to human life. Wholesome air should also contain about 40% of moisture. It is very often necessary to obtain the amount of moisture which is present in any atmosphere, and the process is carried out by an instrument called a hygrometer. There are three kinds in practical use—

The absorption hygrometer.

The condensation " "

The evaporation " "

Fig. 702 shows Saussure's hair hygrometer, acting on the absorption principle, while fig. 703 shows Daniell's condensation hygrometer.

The hair hygrometer shows the presence of moisture, because when it absorbs moisture the hair will contract and move the needle over the graduated scale. In fig. 703 the process is different entirely. Here we have the black bulb filled with ether and the other covered with muslin over which ether is poured. This will very quickly evaporate and cause the ether in the

other bulb to cool. Dew is formed on the outside thereof; at the same instant the thermometer inside will denote the temperature at which the dew was formed, and by means of a table the amount of moisture can be found which is present in the air.

Again, if we know the elastic force at the temperature of the wet-bulb thermometer obtained by a table = f , and the difference of level of the two thermometers = d , the height of the barometer = h , F = elastic force at dew-point temperature, we have the following formulæ*—

If the temperature is above 32° ,

$$F = f - \frac{d}{87} \times \frac{h-f}{30} \quad (399)$$

And if below 32° ,

$$F = f - \frac{d}{96} \times \frac{h-f}{30} \quad (400)$$

Ventilation of Buildings.—The ordinary chimney fireplace will carry off about 29,000 cub. ft. of air per hour. Again, we can find out this quantity with some degree of accuracy by the following equation, if we know

T = temperature of flue gases.

t = temperature of outside air.

D = diameter and L = length of chimney in feet.

V = velocity of gases in feet per second.

$$V = \sqrt{\frac{13DH(T-t)}{D+NL}} \quad (401)$$

N being a constant .02–.06, according as we have clean or dirty flues. Regarding the inlet and outlet ventilation of a room, it is usual to allow 24 sq. in. per head, or 1 sq. in. for every 60 cub. ft. of space; or, again, if they are of the form that warm air is admitted *round* the fireplace, 1 sq. in. for every 120 cub. ft. The inlets should always be placed at or about breathing level, and the outlets actually in the ceiling. The rate at which these two gases will intermix will be inversely as the square root of their densities. One B.T.U. will raise 52 cub. ft. of air 1 degree Fahr. And an ordinary man will give off 284 units of heat per hour, of which 93 are absorbed by vapour, leaving 191 units to heat up the air. A cubic foot of coal gas would produce about 650 B.T.U.'s when burnt.

There are three principal factors in ventilation, viz. diffusion, wind, and unequal weights of air. The first two are of minor importance. In any system of ventilation it is advisable to have more inlets than outlets. But the total sectional area of the two should be equal. To reduce the effects of friction, circular tubes are preferable to square ones; sharp angles must be avoided, and the tubes must be as short and smooth as possible. Outlet shafts are often placed close to flues, to take advantage of the warmth and so get a greater discharging capacity. All joints in the tubes must be carefully made and quite air-tight. Ceiling outlets must be continued right out through the roof into the outer atmosphere. It is the practice of unscrupulous builders to take them into the roof space only, and provide outlets in the gables, even if they take the latter precaution at all. A circular disc should always be provided, as shown by fig. 704, to prevent any down-draughts. Outlet shafts may be taken direct into the chimney, but when so constructed

* Suitable tables will be found in Hurst's *Pocket Book*, pp. 207–208.

they must have mica flaps to prevent smoke blowing back into the room. In new houses it is better to provide entirely separate flues for ventilating purposes. A very simple window ventilator is formed by making the lower rail of the bottom sash deeper than is necessary, and therefore preventing the two sashes meeting closely in the centre. The idea is shown in fig. 705. Regarding wall ventilators, the best practical forms have been found to be Tobin's tube and Sheringham's ventilator. The latter is placed fairly near the ceiling, while the former is carried up the wall, but not usually more than 6 ft., and should not have any shelves above it to deflect the draught. Sometimes perforated bricks are used; they give fairly good results, especially for ventilation under floors, but they must have conical holes. The Tobin tubes spoken of above should be funnel-shaped at the top and at the inlet grating; the latter reason is in order to give an air-way of full bore by making allowance for the natural obstruction of the grating. Ordinary gas may be made to give great assistance to ventilation in the form of either 'Sun lights,' nearly always provided in theatres and music-halls, or the Patent Globe light, an invention of a Mr Hammond.

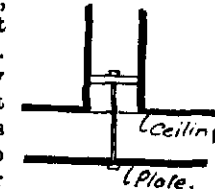


FIG. 704.

The essentials of good ventilation in buildings are—

(1) Each person must have 1000 cub. ft. of space in view of changing the air not more than three times per hour.

(2) Above 13 ft. the height of a room cannot be practically considered as air-space.

(3) Inlets and outlets must be provided, although the chimney often provides for the latter.

(4) Down-draughts must be strenuously avoided.

(5) Shafts must be of smooth bore, and outlets must have a cowl. They must not be longer or have more bends than necessary.

Mechanical ventilation, except in factories and other extreme cases, is unsatisfactory, a natural source of expense, and not to be advised.

The Heating of Buildings.—Heating may be effected either by radiation, conduction, or convection.

Radiant heat is the most healthy, because it will necessarily aid ventilation and does not dry

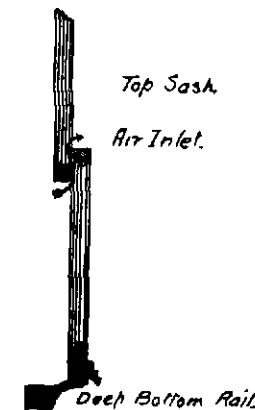


FIG. 705.

p the air, but radiant heat will decrease as the square of the distance. Conduction is the passage of heat from one particle to another, while convection is heat conveyed by moving masses of air, and is the principle of stoves and hot pipes. The design of actual fireplaces demands attention. Here are many common faults in their construction, viz.—

1. The grate placed too far back.
2. Too much iron used in their construction, and hence a large space is left behind.
3. The bars are too wide apart, air being admitted under the fire, which could always be closed by a cast-iron ash-box.
4. The firebrick backing leaning away from the fire, when it should lean all towards it.

Stoves have the advantage of giving more uniform heat, but they have

the disadvantage that they do not cause any ventilation and dry up the air, and are liable to create smells also: carbon monoxide (a poisonous gas) will escape from badly formed joints or be produced by scorching. By their drying up the air, the actual amount of moisture is unchanged, but the relative humidity is greatly diminished, because warm air can hold more moisture before the saturation point than cold. These stoves should have the air supply admitted underneath by pipes, and to prevent overheating they should be lined with firebrick. Gas stoves must always be placed in a fireplace and the chimney left quite open. Regarding hot-pipe heating (the term hot-pipe used here includes either high or low pressure steam or hot-water pipes), the system is absolutely inadmissible unless both inlet and outlet ventilators are provided; which is to say, that it is essential that the air passes in underneath and up through a radiator that has vertical pipes. The supply pipes should never be smaller than 1 in. in diameter.

It may here be stated that we have not attempted to give a treatise upon ventilation and heating of buildings, which is a branch more or less of the architect's profession, but we have tried to place before the reader a few of the principles, because a municipal engineer constantly has such schemes coming before him for his approval, rather than actual design.

Rainfall and Dampness of Sites.—Rainfall is that which causes natural water supply in contradistinction to that obtained from wells, etc. The mean average fall in England is found to be 21.61 in. (we shall refer to this subject again under Waterworks), March being the driest month and November the wettest on an average. The natural cause of rainfall is hot masses of air saturated with vapour becoming intermixed with cold currents of air, and the warmer the air the greater the precipitation; hence the heavy and sudden showers during summer months. The amount of evaporation is usually determined by the soil; an impervious soil, or one which has got very hard after a long drought, naturally giving up more than a porous one. The wetter the land the greater will be the evaporation, and hence coldness of the soil (*vide* Land Drainage). The smaller passages in soil are known as the pores, the larger ones as canals; and in good growing condition the pores will contain water, but the canals air only. Very often no attention is paid to the site of a dwelling-house. It must be dry and warm, and have free access to light and air. Of course the dryness of the site depends on the natural drainage facilities. The best possible building land is gravelly soil, fairly deep, having a small natural fall. But if clay comes under the gravel it will not be so favourable, but by drainage may be much improved. Rock sites are, of course, all that can be desired.

Clay and marl are unsuitable unless very well drained; even then they are liable to be damp.

A chalky soil is dry and very healthy.

Peat land is just the reverse; while made ground, especially old refuse dépôts, are absolutely inadmissible from a sanitary point of view. It is absolutely essential to ventilate the space between the boards and ground-level of any building where there are no basements, and all vegetable earth must be removed and covered with at least 4 in. of good cement or tar concrete. Again, a sandy soil, being absorptive, will always be warmer than one of clay. All damp land must be drained. Regarding the amount of light and air which a house is to get, a south or south-westerly aspect is always to be aimed at. The site must not be too much enclosed by other buildings, nor should it be

* *Absorptivity of Brickwork.*—A newly erected house is necessarily damp until it has been properly "aired," and it is the wiser course not to take a house which has been built in the wet season of the year, unless it has since had a few months of fine open weather, with plenty of sun and wind to dry it. A house built in dry weather will not suffer to so great a degree; but it must be borne in mind that the building materials themselves—the bricks, mortar, plaster, and so forth—all contain a quantity of moisture. Experiments have shown that a rod of brickwork—a rod contains 4300 bricks, and a house contains many rods—when immersed, can, where the bricks are dry, absorb 40,000 oz. of water, while in a similar measurement of brickwork 27,000 oz. of water can evaporate from the mortar. Assuming that the greater quantity of this evaporates—owing to the presence of fires, lights, and the warmth of occupation—into the house, a graphic picture may be drawn of the condition of the premises until the drying process has been completed.

Primary Causes of Damp.—The initial condition of damp is, however, not the worst side of the case, for it at least disappears in time. What the tenant has to fear, in both town and country houses, though not nearly to such a degree in the newer type of the latter, is the constant presence of damp. This may arise owing to many causes, the main among which are inferior brickwork, insufficient ventilation under floors, and the absence of a dampcourse. Taking the question of brickwork first, it need only be said that the bricks of which the house is built should be hard, sound, and thoroughly burnt, while the joints should be absolutely filled with mortar. To watch the piecework bricklayer making his mortar joints is one of the most pathetic sights in the world. He simply dabs one lump for the brick to rest on, adds another along the front edge—and the brick is laid. It is no playing with words to say that, until the final pointing is done outside and the plastering inside, a brick wall may be seen through in scores of places. The experienced surveyor will detect a wall of this kind almost at a glance.

The Prevention of Damp.—Timber floors laid upon the soil, or even upon the concrete, which is now spread to the depth of 6 in. over the site of every new building in districts where by-laws are in force, will inevitably rot unless there is a space for ventilation between them and the soil or concrete. Should there be the least evidence of damp in these timbers, the existence of this air-space should be ascertained by lifting a board here and there, while any damp rising in the walls will undoubtedly be caused by either the absence of a dampcourse or the inefficiency of the one existing. A dampcourse is a layer of asphalt, lead, stoneware, or slates in cement, or other impervious material, laid a few inches above the ground-level, in, and to the full thickness of, all walls. This course prevents the moisture which is absorbed from the ground into the brickwork from going higher. It is essential that all timbers should be above the dampcourse, which means that no wood floor or part of a floor should be less than from 12 in. to 15 in. above ground-level, and the importance of seeing that this is so cannot be overestimated.

House Drainage.

This work must always be carried out under the municipal engineer's supervision. Starting at the junction with the main, which we assume to have been properly formed in accordance with the principles set forth, we have the disconnecting trap. This is shown in section in fig. 706, in

* *Illustrated Carpenter and Builder.*

JOSEPH CORRETT
CIVIL ENGINEER
TOWN HALL.
SALFORD

which we have an inlet for the sewage, and an air outlet which should be brought to the surface by a stand-pipe having a cast-iron grating; there is also a plug for cleaning out to the main, but a better form of trap is that which has a sweeping arm branch, shown by dotted lines. This is brought up to the

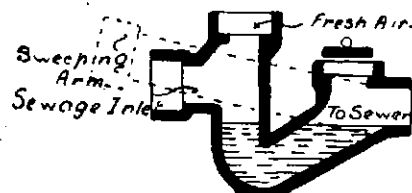


FIG. 708.

surface to allow rods being run down from the surface, in case of chokage and has an air-tight stopper. The traps, if deep, may be conveniently set in a manhole, as in fig. 70. But manholes in house-drain systems are to be avoided, as they only add to expense, and are generally made too small to be of any practical use. It is much more satisfactory to bring the drain at each end of a line of pipes up to the surface and terminate in a sweeping arm, as shown by fig. 708. Coming on to the drain, a 4-in. drain will usually suffice all purposes of an ordinary house drain, 5-in. 6-in. pipes only being used where it is deemed advisable for the purpose of carrying off large volumes of sewage from a large mansion or row of houses.

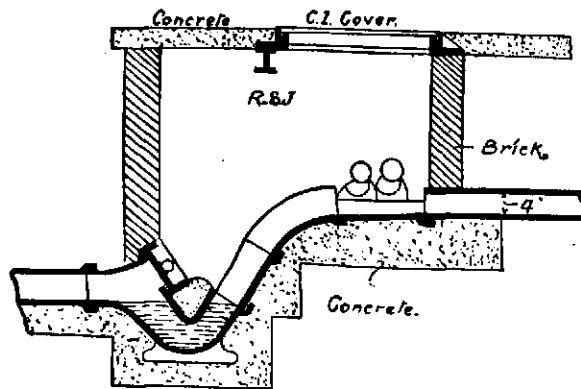


FIG. 707.

The pipes will be laid in precisely the same way as any pipe sewer that is on a concrete bed, and properly cradled and jointed with Medina cement. It may here be stated that there are many patent composition joints now on the market. Their use is somewhat to be advocated, especially in water-logged ground where it would be difficult to make a sound joint. Of the varieties there are many; but, on account of simplicity, Stanford's joint, shown in fig. 709, stands paramount. In the socket is a ring of asphalt composition, and on the spigot end of the pipe a similar ring, but it is made with a barrel surface as shown. These two rings fit into each other, and when wiped over with a special oil and driven home a very good and lasting joint will result, the special shape of the spigot allowing of any irregularity in laying, on the ball-and-socket principle.

* The latest form of this joint is similar to the turned and bored joints on water main, the remainder of the joint being flushed up with Medina or Portland cement. The author considers it the best pipe joint on the market.

When, however, a drain has to go under a house (and such constructions must be avoided where possible), either a cast-iron lead-jointed glass-enamelled pipe must be substituted (of full bore) or the pipe laid as shown by fig. 710, completely surrounded with concrete. Then we shall have junctions to make these, maybe ordinary splay junctions; but where more than one branch pipe can be conveniently brought to the same spot, it is better to use a special form of junction known as Armstrong's junction, of the form shown in fig. 711, and brought up to the surface by a stand-pipe, on top of which is a closed cast-iron cover carefully bedded down in car grease. Where there are,

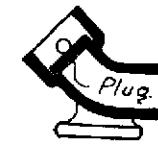


FIG. 708.

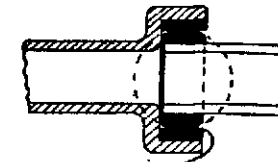


FIG. 709.

however, a large number of junctions, it may be deemed advisable to form a manhole, the sewage being led away in the proper directions by white glazed channels. Cast-iron sealed covers must be provided. Where a change of direction occurs, properly formed bends must always be provided; and if

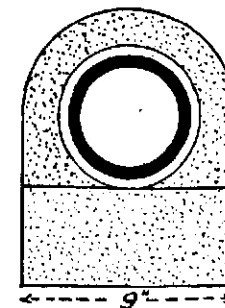


FIG. 710.

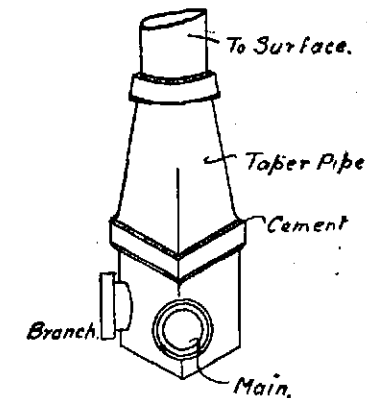


FIG. 711.

necessary, those special forms having an inspection cover provided (Buchan's bends).

Now there will be the surface and yard drainage to deal with. For this purpose the simplest form of gully trap must be provided, made of fire-clay or stoneware, or C.I. set in cement and having a cast-iron grating. A suitable form is shown by fig. 712.

Then we have sinks' waste to deal with. These wastes should be either lead or iron, preferably the latter, with traps (syphon) inside and cleaning plugs. Outside they must deliver into a grease trap by the back inlet, of which perhaps the best known (on account of its simplicity) form is shown in

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CIVIL ENGINEERING PRACTICE.

fig. 713. It has a galvanised iron bucket, whose duty it is to catch the grease and prevent its access to the drains. The admission of grease to drains must be strenuously avoided, because it rapidly hardens in a sort of ring which is continually increasing in size, which may ultimately threaten complete chokage, especially if the fall is flat. The bucket will, of course, require periodic cleansing, a precaution too often overlooked. Rain-water pipes should be of light 3½-in. cast-iron, lead- or putty-jointed, preferably the former. They should deliver into the back inlet of a gully trap in preference to over it, but there is not much to choose between the two. Of course, by placing it over



FIG. 712.



FIG. 713.

the grid the access of leaves is prevented to the drain, but a good deal of splashing may result. Bath wastes and wastes from upstairs sinks, etc., should be formed of 2½-in. heavy iron lead-jointed pipe, and these pipes must deliver under the gully grid by a back or side inlet. The baths and upstairs sinks must be trapped also inside. The waste pipes outside must be carried

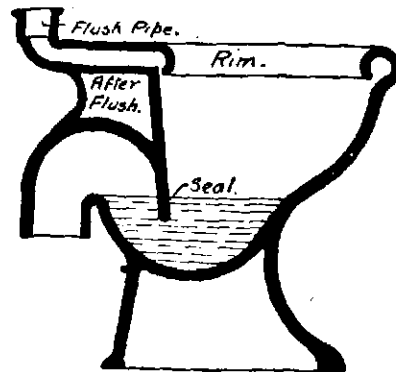


FIG. 714.

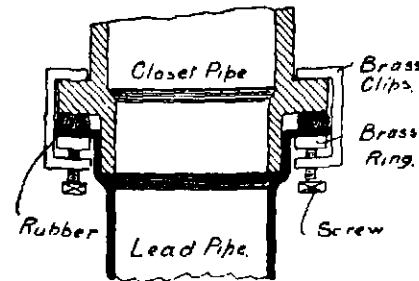


FIG. 715.*

up full bore, 3 ft. above the eaves, and terminate in a bell-mouth with wire netting or a Hunter's cowl.

Lastly, we have the water-closets to take in hand. This is the part of the work which naturally demands the most attention. First of all, the form of closet must be decided upon. The only form for which we have any good to say is the wash-down closet of the simplest possible form, shown in fig. 714, with attendant connections. The material should be of the best possible quality, and the most important joint to have properly made in a whole house-drainage system is that between the closet and the connection to the outer soil-pipe. The whole is shown in full detail in fig. 715. There are

* Mr T. Keatinge, Grafton Street, Dublin, has now placed a very simple and reliable joint for this purpose on the market.

some patent forms also, notably Doulton's metallo-k ceramic joint, which has given very satisfactory results. The closet is finished and fixed in the usual way, and then the outlet is coated with a certain patent preparation, and, again, fixed on to this prepared part, a short length of lead soil-pipe is soldered, the connection between the metal and pottery being so perfect that

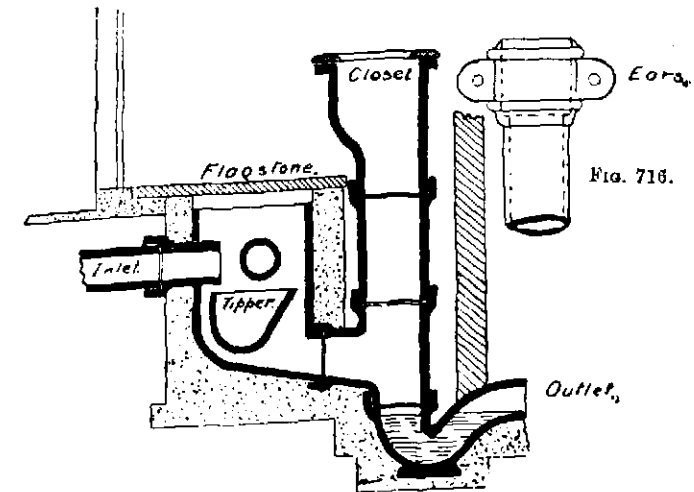


FIG. 716.

FIG. 718.

the lead can only be removed by breaking off the surface of the ware at the same time. The short length of lead pipe is then taken out through the main walls of the building into the heavy cast-iron soil-pipe. This pipe must be at

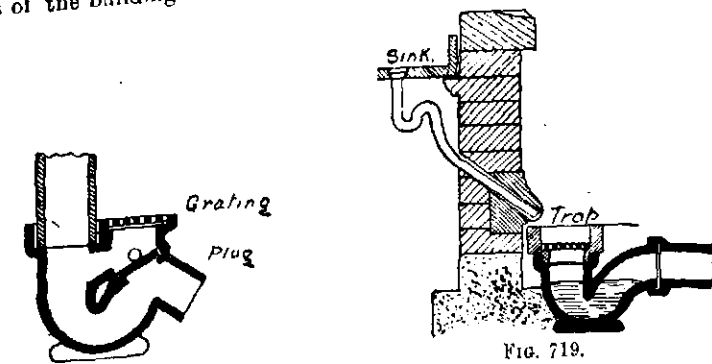


FIG. 717.

FIG. 719.

least 4 in. in internal diameter and lead-jointed, the joints being afterwards caulked. It must be carried up 3 ft. above the main eaves and terminate, similar to the bath waste, in a bell-mouth and wire netting, or a Hunter's cowl. It must be fixed to the walls by ears cast on to each socket, shown by fig. 716. The base should be firmly cemented into an upturned bend of fireclay or stoneware, which is jointed to the pipes, but in some cases a trap at this place may be deemed advisable. Its general use, however, is not recommended;

but in the case of very high soil-pipes, into which a series of closets deliver the whole way down at intervals from the top, such as occur in hotels and big blocks of buildings in general, a big flush in a top closet or slop sink might, by its rush of water, syphon the others, and then the trap will help to deaden the rush of water, and so prevent such syphonage taking place. A special trap for this purpose is shown in fig. 717.

We now proceed to the more important, in this case (that is, in the case of a municipal engineer), matter of testing drains. It is not usual for such an official to actually design drains for houses, but, as before stated, he is in a position to see that such designs are scientifically done and properly carried out according to by-laws, and to investigate bad (or reputed bad) cases where a nuisance exists or is held to exist.

The first thing to do is to sketch a block plan of the house, and mark all the drains, traps, sinks, soil-pipes, etc., as far as they can be ascertained. Having done this, if there is an interceptor trap—that is, a trap which cuts off the system from the main drainage, and so prevents foul air from the sewers entering the house drains—we see if there is a fresh-air inlet to it. If there is one, it should be a 4-in. branch pipe carried to a garden wall, or simply brought up to the ground-level, and a 4-in. C.I. grating on the same;



FIG. 720.

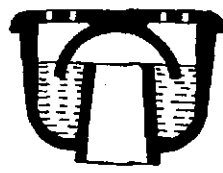


FIG. 721.



FIG. 722.

or the trap may simply have a rising piece to the ground-level, and a grating (not less than 9-in.) thereon. If neither is provided, it will have to be done so. We now ascertain what size the drain is. For a single house, or even two ordinary houses, a 4-in. drain is ample; and although various authorities dispute this, and suggest a 6-in. pipe, I have always specified 4-in., and never had reason to regret it.

The next thing to do is to examine all open traps, grease traps, etc., and none but the simplest forms are at all admissible. The old Bell trap, fig. 721, Antill's trap, fig. 722, the Dipstone trap, fig. 723, and a trap often found at the foot of soil-pipes, known as the D trap, fig. 724, should all be unhesitatingly condemned. Fig. 720 shows a syphon trap. This should also be condemned as unsanitary. Some form of grease interceptor should always be placed as near the scullery sink as possible, preferably on the outside of the wall of the scullery. If this is not provided, the grease will eventually harden in the pipes, and in the course of time will in all probability stop the flow altogether. In fact, it sometimes gets so hard as to refuse removal by ordinary drain rods.

We now come inside the house and inspect the scullery sink. It should be a good fireclay one, white enamelled inside, with 2-in. lead trap and overflow, and deliver into the back inlet of the grease-interceptor trap below the grating. Teak sinks are also serviceable, but lead-lined ones are always a source of nuisance, and should be avoided where possible.

Having satisfied ourselves about the scullery and any other pantry sinks, we look to the house and yard w.c.'s. They should both be good pedestal wash-down appliances, having 2-in. flush pipes, fed from two-and-a-half gallon flushing cisterns. The stinking abominations which one does come across in forms of closets are incredible. The wash-out form is also not desirable, but the hopper-head closet, fig. 725, may be retained (if of a good pattern) or the yard w.c. The bath should in all cases be carefully trapped inside, and if possible be not cased in, and always provided with overflow and safe-tray. This safe-tray should have an open waste in the same way as all w.c. and other cisterns; in no case whatsoever should any cistern overflow or safe-tray waste deliver into any soil- or rain-pipe, for, besides sanitary reasons, the pen end gives alarm when such overflow is taking place.

The soil-pipes and bath waste should now be carefully looked to. The

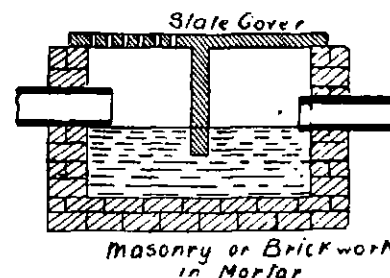


FIG. 723.

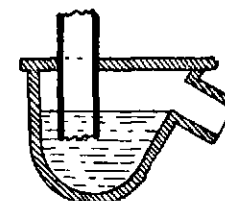


FIG. 724.

former should be a 3½-in. heavy iron lead-jointed pipe, fastened on to the wall by ears, fig. 716, and delivering into a bend at the foot. A trap here, as often so placed by unscrupulous workmen, is quite out of place, except in a special case mentioned before. The pipes should be continued 3 ft. over the main eaves and terminate in a bell-mouth, having a copper-wire setting. Any light iron putty-jointed soil-pipes or bath waste should in all cases be removed. The bath waste should be treated in the same way, except the pipe should be 2-in. only, and deliver into the back inlet of the gully at the foot, and under and not over the grating. If no windows are near, it would be unnecessary to carry 3 ft. over eaves, but it should have a copper-wire netting all the same.

The only remaining thing to examine now is the cistern. In all cases, even when the house is supplied on the constant high-pressure system, a cistern has to be provided for the hot-water supply, but it need not be more than about 170 gallons, which would be in size about 3 ft. 6 in. by 3 ft. 6 in. by 2 ft. 6 in. The cold-water cistern should be either wood lead-lined or galvanised iron. I would prefer the former if the workmanship is the best. A ½-in. service would be provided, and a 1-in. overflow, with open end as above stated. In all cases a cover should be provided. All w.c. cisterns should be fed off the cistern, and the bath cold tap, and also any basin taps; but the scullery sink and pantry should always be direct off the main, and water for drinking purposes drawn from these taps only.

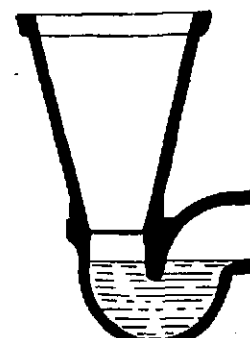


FIG. 725.

Having now inspected all the house fittings, we must set about testing the drain. Obtain a bag or screw plug and insert at the interceptor trap up the drain (never down), turn on the water, and wait till the drain fills and the water rises to one of the traps or access eyes. If the drain holds steady, all well and good; if not, open the ground and see what the joints are made of. They should be of tarred gaskin and either Roman or Portland cement, properly trowelled off. The whole drain must be followed up to find this leakage. Each separate piece must be so treated, as the fall rarely allows of the whole being tested at one filling. Next obtain a smoke machine, place it at the access eye or trap nearest the soil-pipe (if a trap it must be emptied), and pump away till smoke appears at the ventilator. This will show the air pipe is clear. Now plug this and pump away still, and go into the closets and very carefully see if any smoke is getting into the building. Also go carefully into cellars, etc., and search likewise. The efficiency of the smoke test is seen by the pulsations in the w.c. traps, but care must be taken that the water is not forced out of the traps.

Having now got all the possible information as regards the system, the whole must be put together in the form of a report. Before closing this chapter, however, it may be stated that all rain-pipes should have C.I. toes at the end, and deliver into a gully; and also, if no interceptor can be found, by pumping smoke till it rises in the nearest street manhole cover would prove conclusively that such did not exist, provided a flush was sent down the drain to fill it with water, did such a trap exist. On the other hand, such a trap may be leaking so as not to hold any water.

Note in the book whether the outfall is to a sewer, cesspool, or sewage-disposal works.

Carefully examine the inspection chambers, noting if the covers are airtight, and how the chambers are constructed; whether they require re-rendering with cement or not.

The fresh-air inlet should be provided with a mica flap valve, and should be well out of reach of a window. The disconnecting trap should be of a good self-cleansing form, and have a deep water seal. All rain-water gullies should be trapped off from the sewage drains.

The material, dimensions, and construction of all soil-pipes should be mentioned, stating whether carried up above eaves, acting as ventilators. Each vent pipe should be carried up at least 3 ft. above any window.

Note the condition of all gully traps, the depth and efficiency of seal.

State in your book the type of water-closet, if of good design, deep seal, and self-cleansing. This is preferably to be of the pedestal form.

Notes should be made of any earth closets, middens, etc., their construction and sanitary condition. Ash bins should be described as to whether they are portable or fixed, and whether in such a position as not to give off odours to any frequented part of the grounds.

The water supply should next be reported upon, stating whether from the public mains or well. If from a well, the condition of same must be stated, and the water examined to see if there is any suspicion of impurity. The grounds in the vicinity should be traced for any cesspools which may contaminate the water. If there is any suspicion of the water being impure, a sample of it should be taken for analysis. For this purpose use a clean stoppered glass bottle, plunging it well under the water, avoiding surface water and any deposit that may be at the bottom. Rinse the bottle out several times before finally filling, then tie down and seal the stopper. State

on the label before sending for analysis the nature of the soil through which the water passes, the approximate depth of the well, and the distance from any possible source of pollution, such as drains, cesspool, midden, or stables.

The hot-water apparatus of the house should also be examined, and a note made as to the system, whether cylinder or tank, and as to its efficiency for the required purposes.

Reporting on Tests.

It is now necessary to point out how to set down in a proper form of report the information obtained in the last section, with a view to giving the tenant or owner of the property some idea of the state his drains are in, and what repairs would be required to make the system efficient. The first thing to do is to make a block plan (fig. 734) from the sketch referred to on p. 520, as nearly correct to scale as possible. About 10 ft. to the inch (or less) is a convenient size. Carefully mark on this all the traps, soil-pipes, rain-pipes, manholes, etc., also the size and material of each drain, in a similar manner to the plan shown in fig. 734. Having done this, we proceed to report as follows:—

Report No.

Name and Address of Engineer who is making Report

Premises reported upon

On the inst., an examination was made of the above premises with reference to the sanitary arrangements, and I have to report as follows:—

The house soil drain starts from and passes through and onwards to join the public sewer in street (or cesspool or other outlet).

It is formed of and the joints are made with (If it was not necessary to open up the drain, state so here.) It was in working order.

The drains are through ventilated and well planned. (State any other particulars, such as stable drains, etc.) The open traps are of a reliable description, and are clear (or choked). The scullery drain is trapped (or not) from the main system.

The system is trapped off from the public sewer (or cesspool) at by a trap. (State pattern or maker's name.)

The soil-pipe is a one, properly jointed, and carried full bore feet over the main eaves and terminating with (State if cowl or wire netting, etc.) The bath waste is a one properly jointed. It has a ventilator in the form of The bath is a one and trapped inside properly. The waste is trapped off from the system at ground-level by a trap. (State form of trap.) The house w.c. is and the yard w.c. The cisterns are, the flushes are good, and the cisterns have open overflows. They are in good (or bad) working order. The scullery sink is a one, and the pantry sink one; they have properly trapped wastes and the scullery has a grease trap of good (or bad) form. (Describe here any other sinks or slop-closets.) The storage cistern is one and holds about gallons (6½ gallons in one cubic foot). It has a cover and a proper overflow. The draw-off cocks are supplied from (state if main or cistern). Drinking-water may be drawn from the one off the main in (stating any taps direct off the main).

The hydraulic test was applied to the drains with the following results: The smoke test was also applied with results as follows: In order to render the drainage and sanitary arrangements of this house satisfactory it would be necessary to carry out the following recommendations:

The reader will now, from perusing the following pages, see what should be done, and he must use his own judgment in the recommendations he takes.

Where there appears to be no interceptor trap, continual pumping of

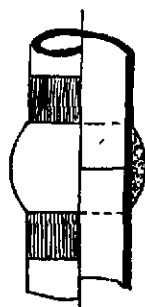


FIG. 726.



FIG. 730.



FIG. 729.



FIG. 731.

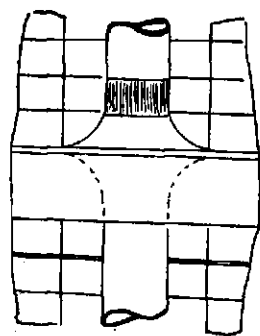


FIG. 727.

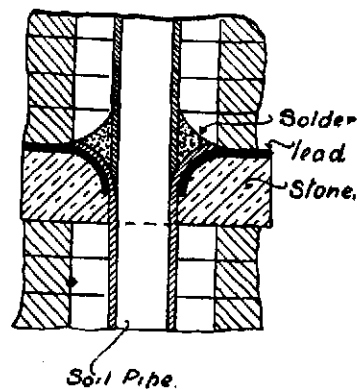


FIG. 728.

smoke into the drains may reveal its absence by smoke rising out of the street manholes.

The Conservancy system which we spoke of consists of drains to carry away the slop water and liquid refuse to some suitable place, and to supply dry closets for the solid matter. With dry closets and privies we are not concerned: the march of civilisation is rapidly, but not too rapidly, putting them out of existence. Another system which has been adopted is the slop-water carriage system where an adequate supply of water for flushing closets is not handy. The apparatus, which at the best cannot be called a sanitary appliance, consists of a tipping trough placed so as to receive the slop water of the house; when this gets free it tips over, and a flush is sent into a specially made closet and washes all the refuse away. The system cannot be advised,

but is mentioned here for the sake of completeness, and is illustrated in fig. 718, a special outlet for the sink waste being shown in fig. 719.

Every form of valve closet is an unsanitary appliance and should be removed.* Very few of these now survive, likewise the wash-out closet. It may here be remarked also that rain-pipes which act as sewer ventilators must be at once cut off and trapped. Bath wastes must not be allowed to enter rain-pipes. Through ventilation of all house drains, no matter how short, must be provided for. Soil-pipes are sometimes made of lead, but this should only be allowed under special circumstances, and only solid drawn lead pipes, 4 in. internal diameter, and made of a thickness = 10 lbs. lead per yard. There are two joints which will suit the purpose, the wiped joint shown in fig. 726, and the flange joint (if deemed necessary) in figs. 727 and 728. The slip joint, 729, blown joint, 730, and the copper bit joint, 731, although the pet fads of unscrupulous plumbers, are not to be allowed under any circumstances. The joint at the foot of such a pipe is shown in fig. 732. Another form of interceptor trap which has not yet been mentioned, known as Broad's trap, is shown by fig. 733. It is suitable for use in a man-hole.

Fig. 734 shows a plan of a complete house-drainage system in accordance with the principles which we have already set forth.

Now it will not always happen that a satisfactory system of main drainage will be handy, in which case some simple and

effective means will have to be provided whereby the domestic sewage may be successfully got rid of. The old-fashioned cesspool, with no bottom and dry rubble sides, together with a chain pump for the periodical and unpleasant process of emptying, has been completely superseded by the more scientific septic tank. These tanks should be absolutely air-tight and as simple as possible, made of brickwork or concrete, and rendered absolutely water-tight: when so designed, and once in working order, they will produce little

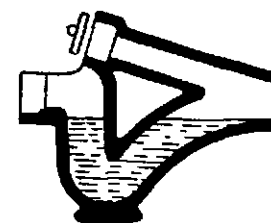


FIG. 733.

or no sludge, and require hardly any attention or cleaning out. A form of tank which is simple, inexpensive, and very effective is shown by figs. 735 and 736. The system has been in use by the writer for many years in private houses and mansions in all parts of Ireland, and has given in every case eminently satisfactory results. It is preferably used in conjunction with a filter drain. The depth of the tank below ground will, of course, be determined by the section of the drain. All other dimensions can be taken from the drawings. The tank should be built of concrete 5-1 ballast and best Portland cement; the space between the walls and

* Some makers are now reviving the use of the valve closet, but with what object the author does not know.

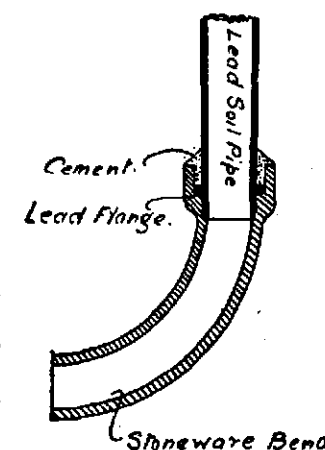


FIG. 732.

the cutting being filled with dry stones, or if handy, clay puddle, and the walls and foundations at least 9 in. thick. The floors should have a slightly curved invert on cross section. The inside of the tank should be plastered with pure Portland cement, finished quite hard and smooth. The covers should be 3-in. flagstones, fitted with lifting rings, the one at the lower end having a hole for the ventilator of the overflow-drain, as shown.

The joint after the pipe is fitted must be left air-tight. Granite corbels

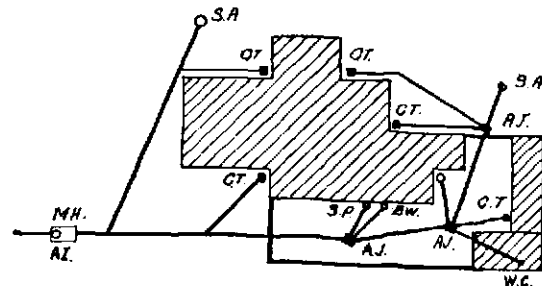
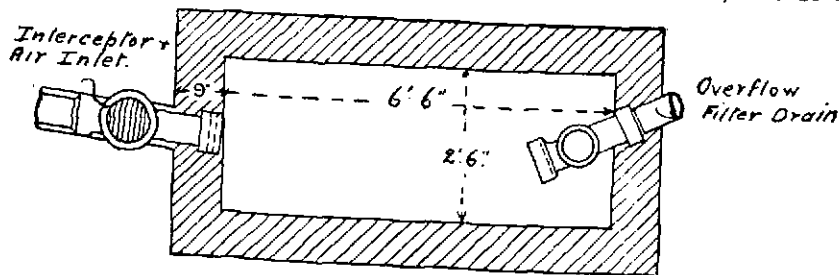


FIG. 734.

The following abbreviations are generally used:—

GT., Gully Trap.	DGT., Deane's Grease Trap.
AP., Air-pipe.	SA., Sweeping Arm.
SP., Soil-pipe.	MH., Manhole.
RP., Rain-pipe.	AI., Air Inlet.
AJ., Armstrong Junction.	BW., Bath Waste.

must be built into the upper end to support a 24-in. square manhole cover, but if the tank has to be deep down it may be necessary to form a manhole down to it, and place the cover at the surface. This, however, will be a



-PLAN-

FIG. 735.

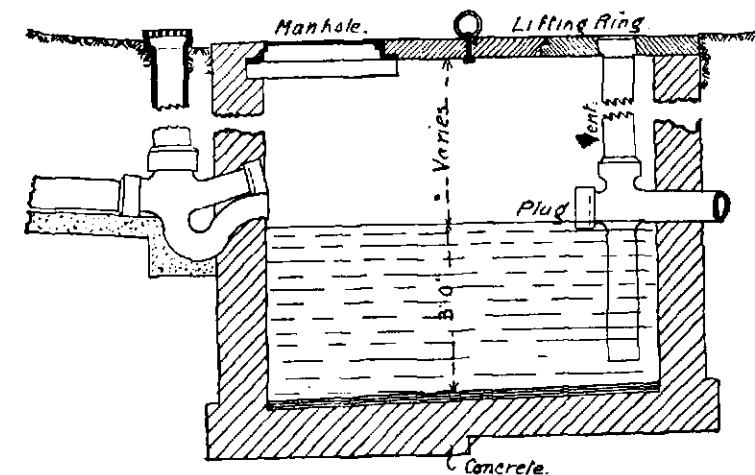
somewhat unlikely occurrence. At the upper end a manhole junction trap must be built in, the outgo invert being 3 in. above the overflow invert. A 4-in. stand pipe and air-inlet ventilator grating must be provided at surface level, with fireclay plugs carefully cemented into the sweeping arms. Step irons should be provided.

The overflow is formed of 3-in. common socketed tile drains (field drain pipes), laid with the sockets downhill. These drains are then surrounded with 3 in. of 2-in. gravel. The drains should be laid fairly flat, say 1-100. Dry

ble cesspits 24 in. square on plan must be provided at each change of section, having flagstone covers, with lifting rings at the surface. They should be laid zigzag and about 1 ft. below the surface, which may be afterwards sodded over. The drains will, however, want careful laying in right lines and with true gradients. Other forms of tank may be designed according to the requirements of the engineer, but they are all practically on same principle, and the simpler the better, besides the material saving of expense.

For the sake of completeness, however, we will present to the reader another type of tank, and leave him to use his own discretion in selection. These types would no doubt be useful under special circumstances.

It is illustrated in figs. 736A and 736B.



-SECTION-

FIG. 736.

Details of Plumbers' Work.

A very usually occurring source of defect in house sanitation is the plumbing. Every care should be taken to see that the plumbing work is properly done, and, as it is usually the small details that are left to the plumber which require the most attention, none but reliable men should be employed in the work. All soil-pipes, especially those of lead, should be placed away from the sun. Lead pipes having seams are, as we stated, absolutely inadmissible, and there is only one joint of any practical value—the wiped joint—made with solder composed of 1 part of lead to 2 of tin. They are held to the wall by what are termed lead tacks. These should be provided, say every 10 ft. or so. The joint with the closet wants special attention (see fig. 715). The work is best done at the works, special care being taken that the rubber ring is of the very best quality and not liable to perish, and that the lead pipe is not weakened by flanging. Special attention is also drawn to the joints in fig. 732, which usually comes under the

heading of Plumbing. The safe-trays under baths, sinks, and closed-in closets (which latter, by the way, are not to be advised, and are rapidly getting out of date) must be carefully made, and the overflow drain requires very careful fixing. It must be made of $1\frac{1}{2}$ -in. to 2-in. pipe, be fixed as shown in fig. 737, and deliver by an open end. No trap is required, but a light copper or aluminium flap is an advantage on the outer end. House sinks should be made preferably of glazed earthenware; those made with wood lined with lead or copper will not give good results.

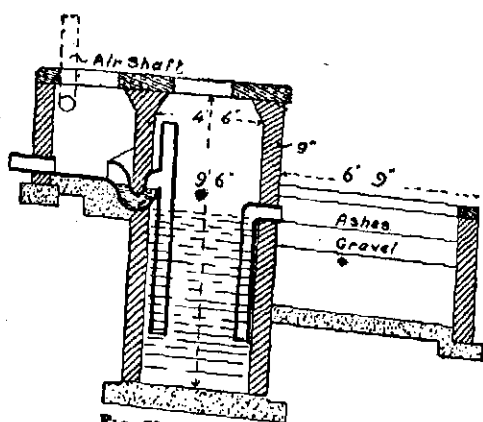


FIG. 736A.—Section.



FIG. 736B.—Plan.

Hard woods without any lining may be used. The lead used for lining should be 8 lbs. in weight to the superficial yard. The waste pipe should be placed in a corner of the sink. On ground-floors the waste and trap

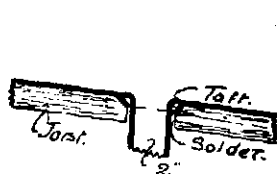


FIG. 737.

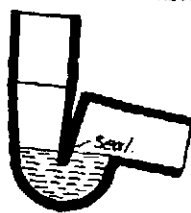


FIG. 738.

should be of $1\frac{1}{2}$ -in. drawn lead or cast iron, delivering into the back inlet of the grease trap, but on upper floors, to prevent syphonage, the trap shown in fig. 738, known as Hellyer's trap, may be provided. It is a very simple and excellent form of trap for the purpose, owing to the bell-mouth inlets and square outlets. The erection of small flushing-tanks is usually left to the discretion of the plumber. We are referring to those tanks for such a purpose as flushing a series of closets in schools, asylums, and other large buildings. It is essential that the pipe be set truly vertical, and the top rim curved inwards. This is so that the first flow of water will fall down the centre of the pipe and cause a good vacuum. If a trap happens to be interposed

between the flushing tank and its place of action an air-break must be introduced somewhere between it and the tank. We before denounced the hopper closet, but we confined our remarks to the long hopper closet, because although we do not advise the general use of such appliances, the short hopper closet may be used in yard w.c.'s, where it would be connected with the drain direct, and so avoid any risk of faulty plumbing. The erection of house cisterns demands the attention of the engineer. They are preferably made of galvanised iron, but where the water has no tendency to eat away the lead, wooden lead-lined cisterns, provided the plumbing is properly done, may be found very satisfactory.* All cisterns without exception must have tight covers. They must be large enough to provide sufficient storage, but not so large as to cause contamination of the water supply. The supply pipes from the main must be carried up internal walls so as to prevent damage by frost, but where they are carried on the inside of main walls (they must never be outside) they must be nailed to 1-in. boards and boxed in, the boxes being filled with sawdust, felt, or cocoa matting. Pipes on roofs should be boxed in likewise, and every precaution taken against frost. Perhaps the most important point of any is the cistern overflow, which must, without exception, be provided. They must under no circumstances whatsoever be connected to soil-pipes, rain-pipes, bath wastes or vent pipes, but must simply deliver by a short length of pipe into the open air, so as to prevent any contamination of the cistern water, and to give warning when they are in use. They must be of larger bore than the supply pipe, and all w.c. cisterns should be similarly fitted. No water for drinking purposes in town houses should be taken from cisterns; a special tap off the main should be provided. The writer has inspected some cisterns in Ireland which would make any ordinary person ill for a week to look at, let alone drink the water therefrom. Dead mice and rats, to say nothing of cats and other putrid refuse, are frequently found in the cisterns of old houses, which have no proper covers.

Infection and Disinfection.—Before concluding our chapter on sanitation, it may be well to say a few words on infection and disinfection, because, although the work is primarily entrusted to the sanitary sub-officers of a town, these gentlemen are usually under the jurisdiction of the borough surveyor, and a few words on their work will, I hope, not be deemed out of place.

Regarding diseases, the first stage is termed *Incubation*, being the period before the disease manifests itself, then comes *Invasion*, and finally convalescence. Patients with infectious diseases must always be isolated completely. The practice of hanging sheets outside the room door, even if soaked in disinfectant, is absolutely useless. The terms "disinfection" and "disinfectant" can only be applied to the absolute destruction of disease virus, the various processes being known to the public under these terms being only merely antiseptic or deodorant. A disinfectant must, to be of any practical value, arrest the growth of, and absolutely destroy, the morbid agent; it must be applied to every part of the affected substance in sufficient strength, and for an adequate period. There are three main methods of disinfection, viz.—

1. Heat (which may be dry or moist).
2. Chemical substances (liquid or powder).
3. Fumigation (by gases or vapours).

No doubt heat is the most satisfactory. It need not be greater than 212° Fahr., and moist heat is preferable to dry, because it has both a greater and

* Slate has also been satisfactorily used.

a more rapid penetrating power, in the proportion of 18 minutes to 8 hours. The machine which seems to give the most satisfactory results is Ransome's hot-air regulator (dry heat). The advantages of steam disinfection above hot air are—

1. Every engineer knows that steam will liberate latent heat. This is, of course, liberated into the articles.
2. In dry heat there will always be a natural loss in evaporating any moisture present in the articles.
3. The condensation of the steam in the articles causes a loss of pressure, and continual admission of fresh steam is essential.
4. Again, saturated steam is in all cases preferable to super-heated.

The most efficient steam disinfecter seems to be Recker's, in which the pressure may vary between $1\frac{1}{2}$ lbs. per sq. in. (216° Fahr.) and 7 lbs. (230° Fahr.), but the cost of the apparatus is high (about £80); and for places where the amount of disinfection would only be small, and not enough to warrant the expense of an elaborate apparatus, Thresh's disinfecter, on the same principle (steam), has given good results; but to obtain a higher temperature than 212° a solution of chloride of calcium is used, because it has the property of having a higher boiling-point than water. Then there is chemical disinfection, the most generally used chemicals being corrosive sublimate (bichloride of mercury), 1-1000 part solution, formalin, cyllin, and chinisol (1-1000 part solutions), and carbolic acid (phenol), 1-20 part solution. Regarding the use of all these, they must be of such strength to come in contact with the germs without undergoing further dilution. Corrosive sublimate is the most powerful of them all. It is very poisonous and must be very carefully handled, which precaution is particularly desirable on account of its natural colourless form. Formalin is the name which is given to a 40% solution of formic aldehyde, which is a gaseous alcoholic compound. In use a 2% solution is sufficient (2 of formalin to 98 of water). It has the paramount advantage of being non-poisonous. It is also cheap, costing less than carbolic acid. Cyllin and chinisol are patent disinfectants. Chlorine is rather useful, especially for washing walls, for which a 1% solution is sufficient. "Chloros" is a solution of sodium hypochlorite, containing 10% of available chlorine. Condy's fluid, though useful for keeping things sweet and clean, is almost too expensive for disinfecting work, because such large quantities would be required. The reason is that the permanganate is used up in oxidising the organic matter in the infected substance.

Fumigation by sulphur is a very satisfactory way of disinfecting buildings. The amount required is about $1\frac{1}{2}$ lbs. per 1000 cub. ft. of space. In use it is placed on a tray in the centre of the room, and a little alcohol poured on to it to give it a start, when it will burn continually till all gone. All windows and doors, etc., must be closed, and preferably sealed with brown paper. Chlorine gas is also used. It may be generated by pouring strong HCl on to chloride of lime in the quantities of 2 pints of HCl to 1 lb. of chloride of lime per 1000 cub. ft. of space. Or it may be produced by mixing one ounce of common salt with $\frac{1}{4}$ oz. of oxide of manganese, plus $\frac{1}{4}$ oz. of H_2SO_4 and $\frac{1}{4}$ oz. of water. Apply the solution to the powder, preferably in a hot basin. But formic aldehyde vapour is superior to both the above processes (sulphur and chlorine). To produce it, paraform tablets are used in an Alformant lamp, or better still, a "Linguer" machine used. To produce "glyco-formal," used for disinfecting purposes in the same way as formalin, a solution of formic aldehyde, glycerine, and water is used. In all cases of fumigation

the rooms must be sealed for at least six hours. When paraform tablets are used, it will be necessary to burn 10 for each 1000 cub. ft. of space, the walls being afterwards painted with a 5% solution of formalin. It may here be mentioned, the formalin has hardly any penetrating power, being merely a superficial disinfectant. But it will not damage metals or colours (except aniline blue). The various actions of disinfectants are as follows:—

1. Metallic salts and mineral acids combine with organic matter, and thus forming a permanent compound, prevent the action of bacteria.
2. Substances such as charcoal condense the oxygen within their pores and help to destroy animal matter. Condy's fluid will give out oxygen and so form CO_2 , while sulphurous acid deprives substances of their oxygen. Corrosive sublimate will be neutralised by proteid nitrogenous matters and albuminoid matter; in such cases an excess over the ordinary quantities has to be used.

For disinfecting typhoid excreta (a very necessary precaution in the case of outbreak) use the following:—

Corrosive sublimate $\frac{1}{2}$ oz. to one gallon of water.

Sulphate of iron 2 lbs. " "

Chloride of zinc 1 quart to 3 quarts of water.

Carbolic acid (Calvert's No. 5) a 5% solution.

To produce nitrous acid gas for disinfecting purposes use $\frac{1}{2}$ oz. copper filings, $\frac{1}{2}$ oz. nitrous acid, and $\frac{1}{2}$ oz. of water. It may here be stated that those compounds termed nitrous contain less oxygen than those termed nitric. Nitre is nitrate of potassium.

For disinfecting stone and tile floors, use sawdust which has been saturated with H_2SO_4 ; and for disinfecting sewage, sulphate of iron only is of any practical value. Very often old offensive burial-grounds have given rise to nuisance. They must be raked over, fresh earth put on, trees planted, and good stiff grass sown. A spreading of unslaked lime is an advantage.

The subject of disconnecting traps is at present a very vexed question, therefore no doubt the views of a very eminent member of the sanitary engineering profession may prove of value and interest to the reader.

At a meeting of the Sanitary Inspectors' Association held recently at Bath, Mr T. J. Moss-Flower, C.E., F.R.San.I., opened a discussion on the disconnecting trap in drains. Mr Moss-Flower stated that he considered it most desirable that there should be fixed on the line of drain between the house and the sewer an intercepting trap, so as to keep the sewer air as far from the house as possible. In his opinion, the fixing in a proper manner of an intercepting trap of suitable size, shape, and form affords an effectual barrier to the passage of any germ of disease or sewer air from the sewer to the house. He had examined the drainage of houses where there was no intercepting trap between the house and the sewer, and the sewer air was enabled to pass freely to the inside of the house, through defective drains and sanitary fittings, and this state of affairs was alleged by medical attendants to have been partly contributory, if not the direct cause, of the specific form of disease from which the inmates were suffering.

He had also examined the drainage and sanitary arrangements of a large number of houses where in each case there was an intercepting trap on the line of drain, and gave particulars of the defects discovered, such as periodical syphoning of traps of w.c.'s, which allowed drain air to enter the house, and which, but for the intercepting trap, would have given a free and uninterrupted passage to the house for the sewer air.

As regards sewers, Mr Moss-Flower's experience teaches him that ventilating openings at the ground-level are the most undesirable, and give rise at least to serious nuisances. Generally speaking, tall ventilating shafts and columns fixed in carefully selected positions are the most desirable; and in places where columns must be erected in exposed positions and close to frequented places or houses, then one of the forms of lamps for cremating the emanations is desirable.

If the size and form of sewers are properly apportioned to the work they have to do, and they be constructed of non-absorbent and otherwise proper materials, be laid with self-cleansing velocities, be regularly flushed, and have ventilating shafts fixed at proper and convenient places so as to prevent undue pressure from rise of temperature and rapid increase in volume of flow; moreover, if manufacturers' refuse, which should be properly treated before being discharged into sewers, and steam and hot liquids above a certain temperature be excluded from the sewer, much will have been done to reduce the nuisance caused by the emanation from sewers.

CHAPTER XXXI.

REFUSE DISPOSAL.

Most towns have some system of disposing of house refuse in connection with the sanitary department. There are various methods in use. In small towns such refuse is spread on waste ground, used for filling up depressions and reclaiming low-lying flood lands, and land from the sea, or taken away out to sea in boats and there disposed of. Another method is the destruction of refuse in suitable destructors, and it is this method with which we are chiefly concerned here. The destruction of refuse by fire seems to date back to the Jewish era, while with the Romans the practice seems to have been discontinued. But one of the most important things in connection with the modern disposal of refuse is the heat energy which can be successfully abstracted therefrom, a practice which also seems to have been in vogue centuries ago for providing heat for Turkish baths in Cairo. The chief reason for the invention of the modern destructor was the difficulties local bodies had in obtaining an adequate area of land on which to dispose of their ever-increasing amount of refuse without nuisance, a necessity which began to be felt about the year 1860. Before going any further it will be well to ascertain what constitutes domestic refuse.

Domestic refuse is naturally principally composed of ashes, to which must be added tins, pots, broken pottery and glasses, paper, rags, bottles and straw. Towns-people should be provided with proper ash-bins. These are usually supplied by the local authorities at cost price. They are then periodically emptied according to arrangement, but some places have large ash-pits which are only occasionally emptied, a practice which is not to be recommended. Such ash-pits would only be cleansed about once a month. The practice is only admissible for hotels, hospitals, and other large buildings.

The best ash-bins are made of galvanised iron, and the carts for removal require particular attention regarding their design, to avoid blowing about of dust and loose paper and bad smells, special sliding covers being the most satisfactory for the purpose. The sorting of the refuse demands attention also. Tin cans are of value for making copperas. Mineral-water bottles, which are frequently found, command sale in themselves, while broken glass and crockery fetch as much as £2 per ton. Again, cast iron is often found and can be made a source of revenue, while people will even pay to be allowed the privilege of picking large heaps of refuse.

But it is the moisture, decaying animal and vegetable matter which make the refuse so offensive and difficult to deal with.

The original furnaces used were of crude construction, and failed principally, it is stated, by the deficiency of flue area, which was less than 3 per cent.

In the absence of a trap on the line of drain, any germs of disease which may find their way into the sewer, from an infected stool or otherwise, may pass up the house drain of a neighbouring house and into the house through one of the numerous defects that may exist in the drains and general sanitary arrangements, and be imbibed by the inmates.

He had never known any evils to arise in connection with the numerous systems of drainage constructed by him during the last twenty years in connection with which intercepting traps were fixed, and of the large number of intercepting traps fixed by him he has only known of one becoming choked, and that was due to the carelessness of workmen in allowing some cement to fall into the well of the trap.

In Mr Moss-Flower's opinion, the abolition of the intercepting trap would prove very disastrous, and it would not be thought of in any case unless the public sewers were constructed, covered, cleansed, ventilated, and kept at all times so as not to be a nuisance or injurious to health, and the house drainage and general sanitary arrangements of all houses draining into the public sewer were also constructed and maintained constantly in a perfectly sound state; and since it is not possible to ensure this, the intercepting trap is the best safeguard to the health of the inmates of houses.

Where complaints have been made against the use of the intercepting trap, they arise from one or more of the following defects:—

(a) The trap is too large or too small; in the former case difficult to flush, and in the latter liable to choke. (b) The trap is of improper shape and form, offering obstacles to the flushing out of its contents. (c) The invert of the outlet of the trap being on a level with, or above the level of, the inlet, in the latter case causing the sewage to pond back in the drain. (d) Carelessness of workmen in fixing the trap out of level, either tipping it towards the outlet, thereby breaking or reducing the water seal, or tipping it towards the house, and making the outlet higher than the inlet, and causing the sewage to pond back in the drain, as in (c); also in not clearing the cement from the inside of the pipes at the joints, etc. (e) By fixing the traps, in cases where there are no special automatic flushing tanks, so far from the house that all ordinary flushes of water have so spread out along the invert of the drain as to have no flushing power by the time it reaches the trap. (f) By giving too little fall to the house drains to which the traps are connected, causing the traps to choke. (g) In the case of disconnecting chambers, by fixing intercepting traps with inspection eyes, leading to sewer, these inspection eyes being capped by plain stoneware disc which can be easily blown out by pressure of air or otherwise, and the stoppers under normal conditions being seldom tight, allowing sewer air to enter the drain. (h) By a variety of other avoidable defects.

The foregoing defects, the speaker maintained, can be avoided if proper care be observed. By carefully selecting the trap, and taking due care in fixing and in selecting the position for it, no evils need arise; and as regards the inspection eye on the outlet end of manholes, the disc closing the opening should be so arranged that it cannot be removed by sewer men thrusting the rods up the house drains from the sewer, or by pressure caused by sewage or air, and yet so as it could, if necessary, be quickly opened from the ground-level without entering the manhole.

Mr Moss-Flower has made detailed examinations of a large number of premises where one or another of such diseases as typhoid fever, diphtheria, blood-poisoning, etc., has broken out, and in every case he has found serious

defects which admitted either drain or sewer air to the house, and to these defects the medical attendant has attributed the disease.

In other cases, particularly in boarding-schools, where there has been no specific form of disease that could be diagnosed by the medical attendant, but a general low condition of health has been prevalent among the inmates, he has found the sanitary arrangements most defective.

In his opinion, it would be necessary, in the event of the intercepting trap being abolished,—

(a) To insist on a very high standard of workmanship and material in connection with the construction of house drains and sanitary arrangements generally. (b) To arrange for a much better system of supervising the carrying out of such work. (c) To arrange for periodical tests and examinations at much more frequent intervals than is usual at present, to make sure of the pipes and fittings being maintained tight and sound, and proof against allowing drain or sewer air to pass into the house. (d) To discontinue* the practice of providing low-level inlet ventilators to drains, as now provided under most local authorities' by-laws, and to arrange for all ventilating openings to be carried to a point well above the roof and away from all windows. (This plan, it was pointed out, is desirable even where there is an intercepting trap, but it would be much more so were the interceptor abolished.)

The speaker's observations, extending over a period of many years, makes clear to him that the low condition of health shown by the inmates of houses where drain or sewer air has access to the house is often to be attributable to the imbibing of such air.

Ventilation of Sewers and Drains.—The ventilation of sewers and drains said Mr Moss-Flower is absolutely necessary, and in the case of house drains—(a) To prevent the air in any part of the drains or pipes from being compressed to such extent (by the discharge into the drains or pipes of such a bucket of water) as would endanger loss by momentum of the water seal of traps. (b) To prevent stagnation of the air in the drain, so that if there were defects in the drains or fittings, the air passing into the house because of them would not be so harmful or obnoxious as would be the case where there is no ventilation. In the case of drains there should be not less than two ventilating openings, as far apart as possible; and low-level inlet opening, now often fixed in basement areas and such places, should not be permitted, low-level inlet only being allowed where it can be fixed a long way from the house, and where the discharges from it cannot be a danger to the occupants of the house or anyone else.

Since one of the objects aimed at by those who advocate the abolition of the intercepting trap is the ventilation of the sewer by means of the soil-pipe continuation to the roof, the point of outlet for the ventilating pipe would have to be selected with great care, and that in many instances it would be most difficult to find a point which would be sufficiently far from windows and other openings into the house to avoid a nuisance and danger to the health of the inmates.

The ventilating pipes from the drains of one house should not be so arranged that, although the outlets may be clear of all the openings into the house they serve, they may be a danger to the inmates of an adjoining house, as is very often found to be the case.

* The author does not agree with this statement, because with outlet shafts only how can thorough ventilation be maintained?

of the grate area. The original practical form of destructor was patented by Mr Alfred Fryer, a Nottingham gentleman, now of the firm of destructor manufacturers of Manlove & Alliot, who are one of the principal builders in the country at the present day. The original Fryer furnace seems to have existed for about ten years, and was put up in Birmingham, Manchester, Bradford, and other large towns. The failure of the early plants was the slow and imperfect combustion, it being found necessary to have independent coke fires to aid the process by re-igniting the gases by aid of forced draught. Then the Horsfall destructor was introduced, which employed a steam jet to effect the same purpose. Of all the destructors at present employed, perhaps the best known is the "Heenan," of Messrs Heenan & Froude, Manchester, fig. 739, known as the twin-cell destructor, the idea, which was patented in 1898, being to have two cells, each charged alternately, and so to alternate the flow of the products of combustion, so that the outlet is always over the incandescent fire, the effect being obtained by a deflecting arch and a single opening in the reverberatory arch.

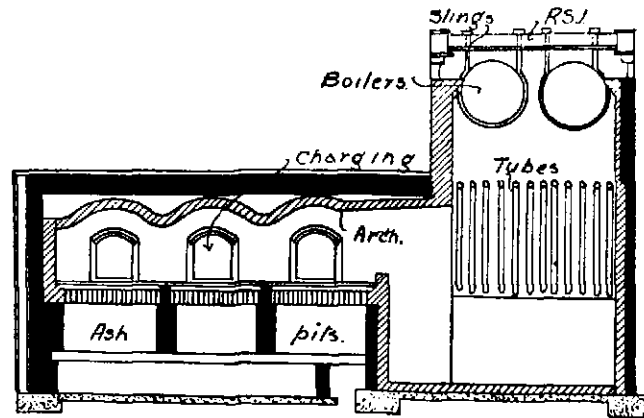


FIG. 739.

Meldrum's furnace, of the simplex type, is shown in figs. 740 and 741, and another favourite type is Beaman & Dea's destructor, while it is only fair to mention the Sterling destructor.

Refuse destructors resolve themselves into two main groups, that is, those having isolated cells, having a number of grates in each chamber, communicating with a secondary combustion chamber by small passages, and those having a primary combustion chamber common to a number of grates having a single outlet into a secondary combustion chamber. Each grate has a separate closed ash-pit. The early forms had only natural draught, which was totally inadequate for the purpose, and the rate of burning has gone up by the use of forced draught to 60 lbs. per sq. ft. of grate area per hour (85 lbs. is said to have been obtained under exceptional circumstances), while the temperature obtained is well over 2000° Fahr. Again, the old furnaces had no steam-raising power, while to-day that item easily reaches 2½ lbs. per lb. of refuse at and from 212° Fahr.

The essentials of a successful refuse destructor are—

1. It must consume or reduce all putrescible and organic matter.

2. The resulting gases must be inodorous, innocuous, and completely sterilised.
3. No dust must escape from the chimney; it must be easily removable and recoverable.
4. A maximum temperature of at least 1300° Fahr. must be attained.
5. The furnace must be kept at the proper heat without the addition of any fuel other than the refuse.

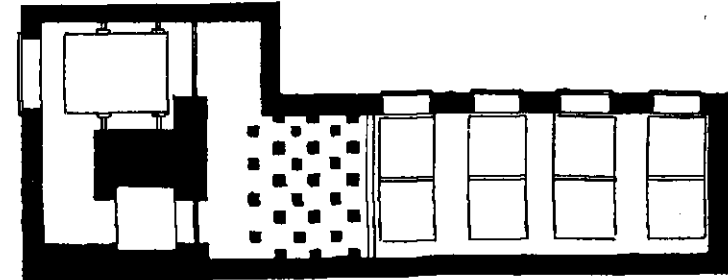


FIG. 740.

6. The resulting clinker must be incombustible, hard, vitreous, inodorous, and sterile.

As before stated, refuse will contain a large amount of moisture, which will be a natural detriment, and may reach anything up to 40 per cent., the other matter being about 30 per cent. combustible and 35 to 40 per cent.

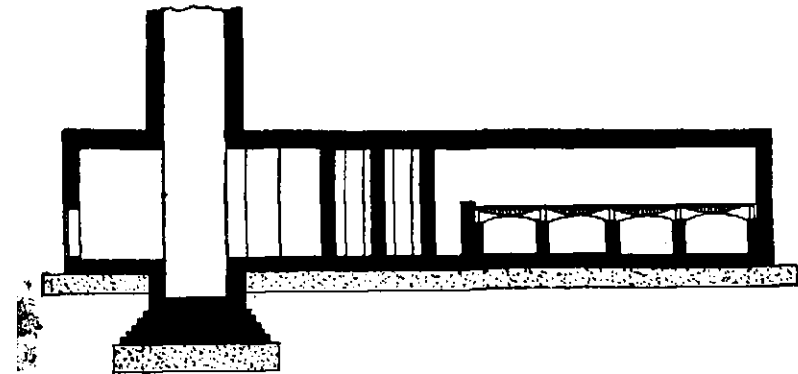


FIG. 741.

incombustible, depending chiefly on the amount of ashes in the refuse, but combustible material is not usually present in sufficient quantity to warrant the use of natural draught. Regarding the actual combustion, the necessary temperature would be about 1600° Fahr.; and although this is above the oxidising temperature of most of the issuing gases, it is necessary to replenish the waste of heat absorbed by the moisture. The issuing gases are generally—

- Marsh gas (CH_4).
- Hydrogen.
- Olefines.
- Carbonic oxide (CO_2).

Again, the higher temperature will reduce the percentage of clinker residue. Thus we see that probably half the available heat in the refuse will be used evaporating the water therein, while the rest will go towards forming clinker, producing the draught, etc., leaving only a small margin for useful steam-raising purposes, and comparing with 1 lb. of coal which would easily under favourable conditions evaporate 8 lbs. of water, 1 lb. of refuse cannot be expected to evaporate more than 1 lb. of water. The proper mixing of gases in a destructor furnace is essential, so as to get rid of as much hydrocarbon as possible, for which purpose the secondary combustion of the Heenan destructor is admirably suited, by producing eddies in the furnace and thoroughly mixing the gases, and it also permits of the gases near the boiler being rich in CO_2 , and having a very low percentage of CO during stoking or clinking, a most essential qualification; and it is for the same reason that increasing the number of grates is an advantage up to a certain limit.

When steam-raising by refuse is considered, the boiler is never quite close to the actual fire, because the rapid abstraction of heat from the more or less damp fuel would cause the fire to die out; and when the space is scientifically proportioned, very little smoke at the chimney will result. Forced draught will increase the temperature enormously, because the pressure reduces the specific heat of the air. It may be produced by a fan or steam jet. Steam has been employed with fair success, but quite 18 per cent. of the actual steam raised will be required to operate the jets; but the particular resulting advantages are—

1. Easy removal of clinker from fire-bars.

2. The clinker is more open in character, but it must not be so friable as to be unsuitable to the purpose for which it is proposed to afterward put the clinker to. On the other hand, with ordinary fan draught, only about 5 per cent. of the steam raised is necessary to produce it; but it has been found that a little steam admitted into the blast is an advantage. It may be easily obtained from the exhaust of the fan engine.

But the use of a heavy blast, and consequent high temperatures, must be considered with care. A high temperature will naturally play havoc with the firebrick lining, causing expense of renewal. The firebricks selected must not split or crack under high temperatures, but should rather gradually melt away. A sort of vitreous protective lining forms thereon after a short time, due to high temperatures, but, again, this must not be so hard and tough as to entail expensive periodical removal. Hence the draught and temperature of combustion demand much careful attention.

The feeding of the refuse into the furnace next also requires consideration. In some places a process of preliminary drying is in vogue. In the writer's opinion, this is not a good practice. It is a matter of difficulty to devise a furnace into which much refuse can be tipped direct without selection, because of the widely different qualities of each cart-load brought to the works. Automatic feeding of refuse destructors has not met with as much success as coal-fired boilers, and hand-firing is generally resorted to.

It may be—

Top feed, front feed, and back feed.

The first named is not generally employed, for the following reasons:—

It requires an opening in the arch, a natural source of weakness.

It would be quite easy for a man to fall into the furnace. The doors get very hot, and gases escape from them, and the proper spreading of the fuel over the fire becomes a difficult matter.

There is no doubt that front feeding is the simplest procedure, because to each grate there is only one opening into the furnace, and there is no undue weakening of the walls.

The process of firing is also very trying to the men owing to the naturally intense heat, and the labour is of necessity very heavy. But with a back-feed furnace, fig. 742, we have probably the best practical form, and that usually now adopted. The tipping floor would be constructed at the most convenient level, and would probably be approached by a long incline for the carts, naturally not steeper than necessary, while the clinking floor is entirely separate. By this system we have frequent firing, which assists in regular combustion and temperature, while it will be quite obvious how much the labour will be lessened. The storage space for the refuse demands attention. It will naturally not be a pleasant process, but good ventilation, with freedom from draughts, will do much to lessen the evil, especially if the refuse is disposed of as rapidly as possible. It may be well to place before

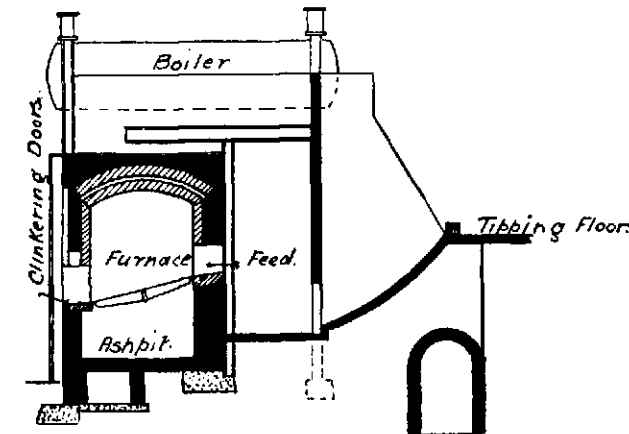


FIG. 742.

the reader some important points to observe when considering the erection of a destructor plant. It is not our intention to give a treatise on design, because that is a matter well left in the hands of specialists, but the important points to be borne in mind when examining tenders and estimates should be brought before the reader.

1. We have already stated, drying-floors are to be well avoided if possible.
2. Forced draught may be considered as essential to successful working.
3. Air is preferable to steam, but it should be preliminarily heated, which heat should be what otherwise would be waste heat, if possible.
4. A large number of grates compared with grate area must be provided for.
5. The combustion space must be ample for complete combustion.
6. The boilers must not be over the flames; adequate space must be provided, so that these flames shall not exist in proximity to the boiler. Regarding boilers, either the ordinary Lancashire boiler or the Babcock or other type of water-tube boiler may be used. Their advantages are stated under Pumping Machinery, but the water-tube type seems well suited to this particular purpose. It is more easily cleaned.
7. A furnace chamber, common to all the grates, must be provided.

8. No undue weakening of the walls should be allowed.

9. These walls should not have an excessive superficial area. The removal of adhering clinker is only an extra expense.

Regarding the testing of destructors, the process is somewhat after the process of boiler-testing, with which we are not particularly concerned here, but the following figures, kindly supplied to me by Messrs Heenan & Froude, Manchester, will doubtless be of interest to the reader.

The description of the actual destructor is as follows:—

Furnace, Boiler, etc.—The destructor proper consists of one unit of 60 tons capacity per 24 hours, containing four grates with divided ash-pits, each grate being served by one clinker-cooling chamber. There is one 183 horse-power Babcock & Wilcox water-tube boiler, without a coal grate; one air heater or regenerator, a fan for forced draught, and such appurtenances as boiler-feed pumps, ventilating ducts, etc.

Carts and waggons enter the property from the street-level and proceed up an inclined roadway to the tipping room, after being weighed on a temporarily installed scale. Carts tip their loads directly into the storage hoppers. Waggons of the bottom dump variety, which have recently been tried, back over the hopper on steel wheel-ways composed of deck beams, plates, and angles. When in position the bottom leaves of the waggon are opened and the load is deposited directly into the hopper.

Refuse is shovelled by hand from the hoppers to the four grates in rotation. This method of hand-feeding seemed desirable in a first installation of this character, as there would be better control over the charging of the material, so that refuse of good quality would assist in burning the poorer kind. After the successive charges of refuse on the grate are burned, a residue in the form of clinker or slag accumulates, which must be removed at intervals, depending upon the amount of incombustible material contained in the refuse. Clinker is withdrawn from the furnace while in a highly heated condition, and drops through trapdoors in the floor to the clinker-cooling chamber immediately below the furnace and on the ground level of the plant; air for combustion is passed through this highly heated material, and returned to the furnace to assist in drying the new charges of green refuse. In this way energy in the form of heat is conserved, and the firemen or stokers work much more comfortably than if they had to wheel away the hot clinkers in barrows, as is done in practically all British plants. After the clinker has been sufficiently cooled by the air passing into the furnace it is deposited in wheelbarrows and removed without difficulty. At present it is used for making land about the building. The provision of the clinker-cooling chambers was made a portion of the original specification for the destructor, and is rather a novel feature of the plant.

The gases from the burning refuse in the furnace pass over the successive grates, mix, and are oxidised in the combustion chamber before entering the boiler. In other words, the furnace is arranged to secure complete combustion before the gases impinge upon any comparatively cool substance, such as boiler tubes. After passing through the boiler, the gases enter an air-heater, which consists of a number of pipes through which the gases pass, and outside of which air for combustion in the furnace is forced by the centrifugal fan. On leaving the air-heater the gases pass to the main flue and thence to the chimney. As there is more or less fine dust contained in refuse, the destructor is arranged to deposit fine materials at various parts of the combustion chamber, boiler-setting, and air-heater, to provide against their escape from

the stack. The furnace contractors spared no expense of time or money in making the destructor the best possible in all respects.

The Tests.—As required by the specifications, a series of tests covering a period of two weeks was started in the early part of May 1908, after the completion of the furnace, and when the brickwork had been sufficiently dried by slow fires. Two mixtures, containing components of refuse representing classes of materials collected during the two extreme months of the year (September and February), were compounded. These mixtures contained the actual weights of materials shown in the table above, which summarises the main features of the five official tests conducted at the plant.

In all tests the flue-gas analyses demand particular attention. The percentage of carbon monoxide must be kept below 1 per cent.; the free oxygen and nitrogen should not have very different proportions than those of the ordinary atmosphere, even quite near the flues. A highest possible percentage of CO₂ (carbon dioxide) should be aimed at, 6–7 per cent. being about a practical average. Even when steam is raised in sufficient quantities to be a source of revenue, we must not forget the fact that the clinker may also be a valuable source of revenue. For instance, builders will very likely buy it to make mortar of, or, again, the steam generated may work a mortar mill, and the actual mortar would doubtless sell at anything up to 6s. per ton. But for the purpose of the municipal engineer, perhaps the most valuable way of utilising the clinker would be the manufacture of paving flags, a process which has been tried and found eminently satisfactory. The process consists of grinding up the clinker and the removal of the iron, of which there is likely to be a large percentage. This is then mixed with Portland cement in suitable proportions, pressed in a hydraulic press, and finally dried; the resulting flagstone would be easily made for 1s. 9d. per sq. yd., and where laid makes a good pavement, which, even if no other advantage accrued, would provide a nonslipping foothold. An excessive quantity may be sold, and they would sell fairly well too, but naturally, before putting down the plant, the demand either for use or selling must be very carefully gone into. They are specially useful for byways and la: s. The bricks made in a similar way have proved excellent, having an average crushing strength of as much as 274 tons per sq. ft. after fifteen weeks. At least 210 tons could be expected on an average. Another precaution to take in reference to refuse destructors is the likelihood of dust being emitted from the chimney, which would naturally cause a nuisance. It is usually avoided by specially constructed dust collectors in the flues. Baffling plates in the flues also have the same effect, but by enlarging the sectional area of the actual flue over a short distance would seem to have quite the desired effect without any other expedient save a few baffling-plates at each end of this chamber; and in order to successfully dispose of the gases coming from a destructor, which at the best of times, even if harmless, are not pleasant, the chimney must be extra high, say 200 ft. a minimum; more especially is this the case in populous districts. Before leaving the subject it would be unfair to pass over Messrs Beaman & Dea's destructor, which is illustrated in fig. 743. It usually works at a somewhat higher temperature than the other forms shown.

Smoke Abatement.

Engineers under sanitary authorities frequently have cases of smoke abatement coming under their notice and jurisdiction, so before closing this

SUMMARY OF OFFICIAL TESTS, WEST NEW BRIGHTON REFUSE DESTRUCTOR.

Test No.	Date of Test.	REFUSE BURNED.				Duration of Test.	Total Refuse Handled.	Refuse Burned.	Amount of Refuse Burned per sq. ft. of Grate per hr.																								
		Description.	Composition, Character.																														
1	May 6	September Mixture as per Specifications	<table><tr><th>Components</th><th>%</th><th>Lbs.</th></tr><tr><td>Garbage</td><td>46.6</td><td>19875</td></tr><tr><td>Fine ash</td><td>21.7</td><td>9255</td></tr><tr><td>Coal and cinders</td><td>7.7</td><td>3284</td></tr><tr><td>Clinker</td><td>0.6</td><td>256</td></tr><tr><td>Glass, metal, etc.</td><td>8.5</td><td>3625</td></tr><tr><td>Rubbish</td><td>14.9</td><td>6555</td></tr><tr><td>Total</td><td>100.0</td><td>42650</td></tr></table>			Components	%	Lbs.	Garbage	46.6	19875	Fine ash	21.7	9255	Coal and cinders	7.7	3284	Clinker	0.6	256	Glass, metal, etc.	8.5	3625	Rubbish	14.9	6555	Total	100.0	42650	8	21,325	20,802	52.0
			Components	%	Lbs.																												
Garbage	46.6	19875																															
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Glass, metal, etc.	8.5	3625																															
Rubbish	14.9	6555																															
Total	100.0	42650																															
2	May 8	Refuse as collected	Wet from rain; Sample dried gave 38% of moisture			6½	16,315	16,145	49.7																								
3	May 13	February Mixture as per Specifications	<table><tr><th>Components</th><th>%</th><th>Lbs.</th></tr><tr><td>Ashes</td><td>79.5</td><td>31881</td></tr><tr><td>Garbage</td><td>11.8</td><td>4732</td></tr><tr><td>Rubbish</td><td>5.3</td><td>2125</td></tr><tr><td>Glass, metal, etc.</td><td>3.4</td><td>1364</td></tr><tr><td>Total</td><td>100.0</td><td>40102</td></tr></table>			Components	%	Lbs.	Ashes	79.5	31881	Garbage	11.8	4732	Rubbish	5.3	2125	Glass, metal, etc.	3.4	1364	Total	100.0	40102	8	20,051	19,827	49.6						
			Components	%	Lbs.																												
Ashes	79.5	31881																															
Garbage	11.8	4732																															
Rubbish	5.3	2125																															
Glass, metal, etc.	3.4	1364																															
Total	100.0	40102																															
4	May 15	Refuse as collected	Wet from rain of previous day			5½	17,430	17,235	62.7																								
5	May 16	Refuse as collected	Relatively dry, representative material			8	23,847	23,673	59.2																								

Test No.	RESIDUAL.					EVAPORATION PER POUND REFUSE BURNED.				
	Clinker, Lbs.	Ashes, Lbs.	Dust (Approx.), Lbs.	Tins, etc. not fired, Lbs.	Total Lbs.	Percentage of Original Refuse.	Gross Actual Lbs.	Gross Equivalent from and at 212° F., Lbs.	Net Useful Steam for Power from and at 212° F.	
1	10830	787	426	1048	13189	30.9	1.17	1.41	1.31	
2	8390	787	326	340	9843	30.2	1.03	1.25	1.16	
3	41466	1978	401	448	14293	35.6	1.10	1.33	1.24	
4	12955	669	349	389	14372	41.2	0.91	1.10	1.02	
5	17344	913	477	349	19083	40.0	1.00	1.21	1.12	

Test No.	CO ₂			TEMPERATURE IN °FAH.				Average Steam Pressure in lbs. per sq. in.	No. of Fires Clinkered.	Average Time per Clinkering Min.
	Average. %	Maximum. %	Minimum. %	Combustion Chamber.		Chimney Gases.	Outside Air.			
				Average.	Maximum.					
1	12.2	17.0	6.0	1846	2210	398	48.5	137.4	9	9.0
2	12.3	16.5	8.0	1715	1922	380	51.5	139.2	8	8.4
3	12.5	17.0	6.0	1637	1940	364	83.9	130.5	10	11.9
4	12.4	17.6	8.6	1698	1904	397	50.6	136.4	7	12.3
5	12.8	16.3	7.6	1792	1940	...	...	137.4	5	8.2

chapter a few words might be of interest to the reader. The remarks apply rather to steam generation than domestic fires, including, of course, a manufacturing works. Let us first investigate briefly what goes on in a furnace. During combustion, coal undergoes certain chemical changes and smoke is produced. To understand fully these changes the reader would require to know something about the properties of coal, which we are not prepared to discuss fully here. Briefly, however, coal which is used as a fuel for steam-raising usually produces smoke (hard coals such as anthracite do not usually find place in furnaces, but are much used in power gas plants). The coal referred to usually contains about 80 per cent. of carbon, together with hydrogen, nitrogen, oxygen, and sulphur, and some earthy impurities.

When coal is placed on a fire, a distillation process takes place similar to that which takes place in a gas retort. During this time the coal is being decomposed and evolving gases, notably hydrogen, marsh gas, CO_2 , and olefine together with watery vapour, these gases all being combustible. Now combustion will only take place when the heat generated is sufficient to evolve

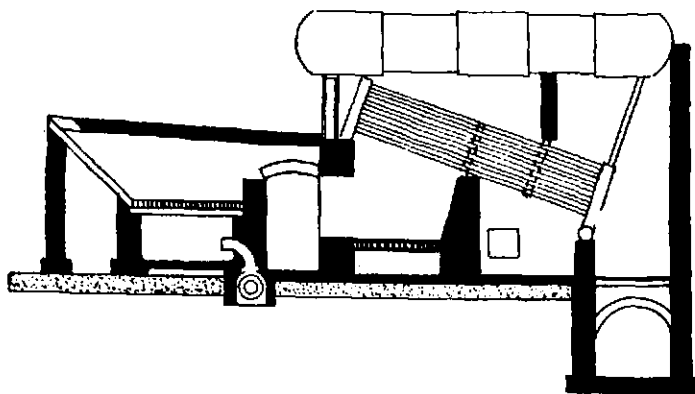


FIG. 743.

light, and is complete when the residue is CO_2 and water. A pound of a fuel requires for its complete combustion 8 lbs. of oxygen, and as air contains 23 per cent. of oxygen by weight, a fuel which contains 4 per cent. of hydrogen will require 4×348 lb. of air for its complete combustion; and if C = the percentage of carbon, the weight of air required will be

$$W = (C \times 116) + (4 \times 348) \quad \dots \quad (1)$$

and to reduce this to cubic feet divide by .067. Call this quantity A .

Then if a = weight of ash remaining, t = temperature of the flue gas above that of the atmosphere, and w = weight of fuel, H = heat carried away in flue gases,

$$H = (A \times w - a) \times .26t \quad \dots \quad (2)$$

Smokeless combustion will require a high temperature (1240° Fahr.) to secure ignition, and also sufficient air to combine with the carbon and form CO_2 . Now the hydrogen in the fuel will combine with the oxygen of the air and form H_2O = water; therefore if there is insufficient oxygen to form CO_2 also, or if the temperature falls below 1240° Fahr., then soot and smoke will be deposited; hence we have a temporary emission of black smoke when

fresh coal is put on a furnace because, due to the thicker fire, more air is wanted and less gets in, while the opening of the fire-door will reduce the temperature of the furnace walls, but the opening of the louvres in the door immediately after putting fresh coal on will mitigate the evil. The foregoing principles will also show the reason for the favourable adoption of mechanical stoking.

Besides the gases before named, we have ammonia, nitrogen, and sulphur compounds which are given off, and more or less combustible. It will therefore be seen that the actual coal does not really burn, but only the gaseous compounds, leaving the solid residue, ash. Now, during practically good combustion (perfect combustion in boilers is impossible), besides CO_2 and H_2O we have nitrogen and H_2SO_4 evolved, but to ensure the best results the boiler furnaces must be scientifically designed for the particular fuel they are to use, a subject left in the boiler designer's hands. Suffice to say, it is remarkable how mechanical engineers, who have so successfully perfected the steam engine, still pay little attention to improvements in boiler design.

Now, issuing from a chimney are finely divided particles of carbon, that is, the visible part of the smoke, but they are usually mixed with very large proportions of watery vapour, CO_2 , and nitrogen, the actual carbon not being more than 3 per cent. of the issuing gases; hence it will be readily realised that boilers and flues which are so designed as to give practically smokeless combustion will result in economy to the owner. Now it is important that the engineer should be aware of the fact that fogs are not caused by smoke by reason of this finely disseminated carbon which it contains, but by the sulphur in the coal, which will finally oxidise to H_2SO_4 ; the carbon has nothing to do with it; their prevention seems to lie in the almost impossible process of prohibiting the formation of H_2SO_4 . The patents and designs put forward for burning bituminous coal without smoke are legion, we may say. There is no doubt, however, that proper boiler design and careful and experienced hand-firing or mechanical stoking will do much to reduce the smoke evil, and the laws which an engineer to a sanitary authority can, by reason of his office, enforce, should be rigidly enforced, because, under ordinary circumstances, black smoke is preventable; but the particularly necessary and often difficult part of that official's duty is to decide if the circumstances of a certain case are ordinary, and that action is justifiable.

CHAPTER XXXII.

WATERWORKS.

NOWADAYS the existence of a pure and sufficient supply of water is essential to every town of any importance. Pure water is as important to our well-being as pure air, and to-day engineers are constantly striving to meet the ever-increasing demands of the population, both at home and abroad, for an unfailing supply of pure water. That such works are generally brought to a successful issue is made apparent by the accounts of new works opened which appear almost daily in the papers. In fact there is no doubt that many years ago towns sprang up on sites which among their other qualifications had a good water supply, which had it not been present would doubtless have resulted in very few settlers remaining there. People, of course, in the olden days, went to where the water was easily obtained, whereas now it is the duty of the engineer to bring the supply, no matter what the distance, to the people; and it is the process of waterworks construction and water distribution adopted which is about to be described as concisely as possible in the present chapter.

The actual design of waterworks, from inception to completion, if they are of any size, is naturally a piece of work which demands much forethought, judgment, and experience on the part of the engineer. In fact, the administration of the waterworks of a town, together with the sewerage system and roads, form the principal branches of a municipal engineer's practice.

First of all, let us consider the water from a sanitary point of view, because if a prospective water supply be not pure enough for the purpose of domestic distribution, and is incapable of purification, any works in connection therewith would only be waste of time and money.

Water in chemical composition is written H_2O , that is to say, by volume it is composed of 2 atoms of hydrogen to 1 of oxygen, but by weight it is 1 of hydrogen to 8 of oxygen. In the state in which it occurs it contains various substances, both in suspension and solution, some harmless, others injurious, and some actually beneficial; but if the water is to be considered pure and wholesome for ordinary purposes, the total solids should not exceed 8 grains per gallon, and in no case 30 grains per gallon. The two main sources from a sanitary point of view of water for domestic purposes are—

1. Rain-water, which will naturally contain a fair percentage of ammonia, together with a little organic matter.

2. Water from lakes, rivers, and wells, which will contain a larger or smaller quantity of salts in solution, vegetable and organic matter, may possibly be contaminated by sewage products.

Water which becomes poisonous when in contact with lead pipes must be very carefully considered. We shall return to the question later.

The presence of infusoria would not be considered injurious, nor would a small amount of organic matter or a *very* small quantity of nitrates or nitrites be very bad, but the presence of these latter impurities, together with the fact of the water having very little saline matter, would tend to increase its solubility for lead salts: $\frac{1}{2}$ of a grain per gallon of lead in solution is injurious. Water which contains chlorides will act upon iron, especially when it is galvanised. On the other hand, if there is lime in solution also, the effects will be neutralised, and the water may be safely used. Of course, when a large water scheme is in consideration, the same as a sewage disposal works — only perhaps the present case even is more important — the services and advice of an expert chemist are absolutely essential; but the engineer can satisfy himself in the first instance, or in the case of small supplies, as to the suitability of a certain supply by the following simple tests:—

1. To test for the hardness of water, dissolve pure white soap in alcohol and add the solution to hard water. The result is a milky-white solution.

2. To test for the presence of copper add some sal ammoniac and iron filings to the water, and a blue colour will result.

3. To test for the presence of carbonic acid, add lime water (liquor calcis) to the supply, and a milky solution will result. The lime water may be made from slaked lime and water, the solution containing $\frac{1}{2}$ grain of oxide of lime (CaO) to the ounce.

4. To find out if sulphur is present, allow some mercury to stand in the water, when its presence will cause a dark surface to appear on the otherwise silvery mercury.

5. Lime in solution will cause a milky precipitate on the addition of oxalic acid.

6. Sulphate of lime. The addition of barium chloride causes a white precipitate.

7. The presence of alkalies (which will neutralise any acid present) will turn litmus paper blue if it has been reddened by dipping in an acid solution such as vinegar beforehand.

8. To test for the presence of iron, add a solution of potassium ferrocyanide to the water, which will cause the natural blue colour of the ferrocyanide to disappear.

9. The presence of lead will cause a black discolorisation when ammonium sulphide solution is added to the water.

10. The presence of magnesia (which is of more importance in boiler-feed water than domestic supplies) is proved on boiling the water and adding ammonium carbonate and phosphate of sodium. The result will be a white precipitate of magnesia. It may here be stated that too much hardness in water, besides being a commercial disadvantage, is also very unpopular amongst domestic users. The principal cause is the presence of bicarbonate of lime. It is removed more or less successfully by adding lime in solution. This will combine with the CO_2 and become chalk, which will settle. A degree of hardness is 1-70,000 parts of the mineral causing the hardness. Water which has 5° of hardness and under may be considered a soft water, and over 12° a hard water. Regarding the effects of filtration of water (we shall proceed to describe the methods later) it has been found that efficient

large,—anything up to 80 in. per annum may be expected; but not so on the leeward side, the rainfall there being very small, especially those places right at the foot of the hills. Dublin is situated in a small basin east of the Wicklow mountains, and has the lowest rainfall in Ireland from the above cause alone. It may be generally stated that in hilly districts the rainfall will increase as the elevation, but only up to a certain limit. The reason is that the mountains cause the air currents to rise, and so cool them. Again, the intensity of the fall of rain will decrease with the height, but the duration will be longer. Again, the temperature will fall naturally as the ground rises, usually 1 degree for every 328 ft. in height. In England the rainfall will increase up to 2000 ft., and then rapidly fall off. It is these conditions which cause the abnormal rainfalls in the English Lake district, and in the county of Kerry, Ireland. Seathwaite in Cumberland can boast of the heaviest rainfall in England. Now, by a long study of rainfall in England, Mr G. J. Symons, whose *British Rainfall* is the *only* reliable record to which we can turn for information, arrived at the following conclusions:—

1. The driest of three consecutive years of dry weather will have a rainfall of $\frac{1}{3}$ less than the mean.
2. The driest of two consecutive dry years will have a rainfall of $\frac{1}{2}$ less than the mean.
3. The driest year will have a fall of $\frac{1}{3}$ less than the mean.
4. The wettest year will have a rainfall of half as much again as the mean.

And it has also been proved by other engineers that if an average of 20 years' rainfall is taken, and $\frac{1}{3}$ of the average is deducted, we shall have, within a very near approximation, the average of 3 minimum dry years, a quantity which for waterworks purposes can be relied upon as the probable rainfall. The amount lost by evaporation will now demand attention. It will vary from 10 in. to 18 in. according to the surface formation.

10 in. is rare; only on hard, glassy, non-absorbent rock such as slate would it occur; with soft soil and peat the conditions are different, and the mean loss by evaporation is now usually taken by engineers, under ordinary circumstances, as 14 in. Experiments have been successfully made upon percolation, and it has been found that about 23.4 per cent. of the rainfall may be expected to be lost by percolation through 3 ft. of soil, and 38.6 per cent. through 3 ft. of chalk. Chalk has the peculiar property of a great avidity for moisture. Then summer and winter will also control the figures; more loss will take place obviously in the summer than the winter. Forests also have their effect on the rainfall. They are likely to augment it, and at the same time they cut down the loss by evaporation to $\frac{1}{3}$. It must here be impressed upon the reader that when rainfall records are looked into they must have every evidence of being accurate, otherwise they are worse than useless; those over short periods only are of no value whatsoever; only by patient attention can accurate records be obtained. The rain gauge itself wants very careful attention, too, in designing. There are various forms, one of the most simple and effective being shown in fig. 745. The instrument is usually made of copper, and the top of the cone is of known sectional area. It must be set truly horizontal, and at such a height above ground as to avoid any effects of

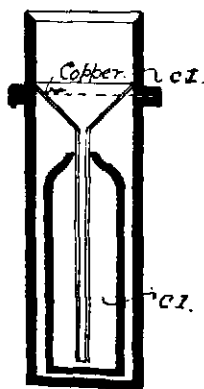


FIG. 745.

splashing. The lower receiving-chamber must be below the effects of evaporation. The contents of the gauge must be measured by a special accurately graduated measuring-glass; 5 in. diameter is the usual size, and 12 in. above ground a suitable height to place it. It has been found that when gauges are placed at considerable elevation the readings will differ from those near ground level, and would not provide accurate results for waterworks purposes. (Automatic self-registering rain gauges are also in vogue.) Nor must they be in any position which is sheltered by buildings or trees. All observations should be made at least once a day, and at the same time of day. Any snow which has fallen into the gauge may be melted and recorded as rain, but it is not likely very accurate results will occur; still, they will err probably on the side of safety, except, however, where the snow would be piled up in the funnel by wind.

It is useful in making such calculations to remember that 1 in. of rainfall over 1000 acres in 1 day will be nearly equal to a discharge of 42 cub. ft. per second. Again, 1 in. of rain per hour = 1 cub. ft. per second discharge off one acre. It must not, however, be supposed that one rain gauge will suit a large watershed. Several will be required, because great difference in the rainfall may be experienced, due to local conditions. It may here also be stated that the bevelled edge of the rain gauge demands particular attention. Its duty is to cut off the drop of rain which should not wholly fall into the gauge. It must be very sharp and slope inwards. Then when the actual rainfall has been arrived at, it is necessary to find out how much we shall be able to induce to come to the storage reservoir. First of all, we have loss by evaporation and absorption. We have discussed it and set the average at 14 in. Secondly, we have to consider the maximum period of time over which the demand will exceed the supply. That is, what the storage reservoir has to supply. Again, the fluctuation of the daily demand will require attention when we come to settle upon service reservoirs. Now we know that at subterranean depths there are great bodies of water, the earth being saturated thereby. The water which falls on the surface falls down practically vertically to this surface, called the "saturation" surface, and is then deflected by it to a certain slope. This slope will vary, being flat with hard rocks and chalk, gradually increasing as the subsoil attains a softer nature. It is also generally inclined towards the sea-level. Then, again, we have what is known as the *free level*. It will usually coincide with the saturation surface, but not necessarily so. Now, when the saturation surface rises to the ground-level we have waterlogged ground, and when such ground has a slope of less than the hydraulic gradient (*vide Hydraulics*), which would cause the water to flow through the earth, we have *surface flow*. It is to this cause then (that of a high saturation surface) that our rivers and watercourses exist, in the form of surface water.

Sources of Supply.—There is no doubt that surface water is the best form of domestic water supply. Then come wells. These must be sunk into water-bearing strata, because they depend mainly upon percolation for their supply. They, in fact, intercept the saturation surface. Where this surface suddenly rises to the surface of the ground, we have what are termed intermittent springs. But this water may disappear down "pot" holes or "swallow" holes, due to the saturation surface also disappearing. These springs will depend entirely upon the rainfall, and the quantity obtainable can be measured with some accuracy, but the water issuing therefrom is very likely to be hard, iron being usually present in about 1 to 500,000 parts

filters will rid water of from 90-95% of micro-organisms. Their efficiency will depend upon—

- (a) Ample storage to allow the bacteria to perish.
- (b) The thickness of fine sand.
- (c) The rate of filtration.
- (d) The frequency of renewal.

Filters, it must not be supposed, are only for the purpose of relieving water of its suspended matter—the screens at the waterworks will do a good deal of that—but an equally important effect is that due to the vital actions of the living organisms, which is assisted by exposure to sunlight and the natural separation of the water into minute streams, that facilitates the oxygenation of the organic matter which may be contained in it. Therefore it is obvious that water should be allowed to filter as slowly as possible, and the storage reservoir should come before the filters; but if this is not deemed advisable, the service reservoirs must be covered. The maximum speed of filtration should not be more than 6 in. to 9 in. per hour, upon which is based the superficial area of the filters, the material usually being from 3 ft. to

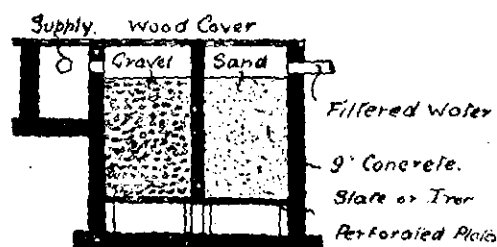


FIG. 744.

5 ft. thick according to circumstances, $\frac{2}{3}$ being sand and $\frac{1}{3}$ gravel, the size of the material increasing uniformly from the top downwards. The size is usually based on filtering 100 gallons per superficial foot in 24 hours. Fig. 744 shows the arrangement which would serve a small domestic filter. The drawing is self-explanatory, and we are going to treat filters for large water-

works in a separate paragraph; but as this small installation is more of a sanitary nature than actual engineering, we insert it here. Small portable filters used in houses are more or less family fads, and at the best of times cannot be said to be much use. In fact, they are often in such a state as to be positively dangerous. They usually consist of porous pots, but these fail when lead and other poisons are present, and charcoal filters have to be substituted.

Nowadays, with up-to-date water supplies, the use of such contrivances is a supererogation in dwelling-houses, but of course in out of the way places, where the supply is doubtful, such as would be the case with an army camping out, their use is somewhat of a necessity; even then boiling doubtful supplies may be said to be as safe a precaution as any. In charcoal filters, however, animal charcoal must not be used, because it is liable to oxidise the organic impurities, and by so creating phosphates and nitrogen will favour the growth of bacteria. Good filters of this type will decompose organic matter in solution, but not so the fresh organic matter. The only household filters of any real practical value are the "Berkfeld" and the "Pasteur Chamberland." The former has a cylinder of diatomaceous earth called "Kieselguhr," which is porous but very brittle, while the latter has a cylinder of fine-grained unglazed porcelain. The Berkfeld filter works under pressure and the Pasteur by gravity alone. To sterilise suspicious water 15 grains of bisulphide of soda to the pint may be used. The essentials of a good water supply may be summed up as follows:—

The water must be limpid and entirely free from smell. Vegetables should be quite capable of boiling in it without discoloration, soap should be dissolved without curds. Nitrate of baryta* should only slightly affect it, indicating the presence of sulphate in combination, and likewise nitrate of silver, which indicates the presence of chloride and sodium. On evaporation there should be very little residuum. A small percentage of CO_2 is an advantage in a water supply. The organic matter can be detected by gallic acid or chlorine solutions; waters which contain much bicarbonate of lime or hydrous oxide of iron will deposit these impurities in cast-iron pipes and supply-conduits.

Regarding a rain-water supply which is used in country-houses, it must be used with some care for drinking purposes, and boiling in all cases is advisable, although the impurities are principally organic. When calculating on such a supply each square foot of roof area should yield 13 to 15 gallons per annum easily. Water of this kind always contains much carbonaceous matter, which it obtains while passing through the atmosphere. In fact water is remarkable for its power of solution; hence the original source of a water supply must be very accurately determined because polluting agencies, not always visible at first sight, often provide extreme difficulty of determination. And, besides, it is not always the case that because certain salts and gases are present in water it will be directly impure, but because these particular elements will give opportunity for organic pollution. It is what are termed *algæ* which will flourish in light and nitrogenous compounds. These, in turn, help to sustain the growth of bacteria and fungi; therefore the presence of nitrogen is a most important consideration. Again, chlorine, which we said is often present, may or may not be injurious. It may cause one to be suspicious of sewage pollution. Albuminoid ammonia must not be present in more than one part in ten million. The taking of samples for analytical purposes demands great care—more, in fact, than is sometimes bestowed on it. Glass-stoppered bottles must be used of at least one quart in capacity. They must be rinsed out a few times with the actual water in question, and immediately sealed when the sample is taken, dated and labelled. Looking on the water from a biological point of view, there is no doubt that water is the direct means of conveying some zymotic diseases, especially typhoid and Asiatic cholera. Again, temperature and light are important factors, and the aeration of the water in its passage over falls and weirs or other places where natural disturbance of the main stream occurs. We have advised the services of an experienced chemist as most essential, but it may here be stated that, fortunately, when on a superficial examination it has been found that the water supply proposed is quite suitable, it is not very likely that the further investigation will prove the presence of sufficient foreign matter to condemn the supply as totally unfit.

Rainfall.—That all water supplies depend primarily upon the rainfall is obvious to all. In England the chief source of the rainfall is the Atlantic Ocean and the prevailing westerly winds. Watery vapour, produced by the sun being about $\frac{1}{3}$ of the weight of atmospheric air, tends to rise, and collects together in clouds. This vapour is not visible to the eye at first, but it is when partial condensation has taken place that they take the form usually known to the public as clouds. When these clouds strike the hills, these hills act as huge natural condensers, with the result of a large precipitation of moisture. On the windward side of such hills the rainfall will naturally be

* Baryta is the heaviest of the alkaline earths.

Again, we have river water. This supply is obtained from the tributaries direct rain, and percolation. But in Britain river water is not suited to domestic supply, owing to the great risks of pollution, except where it is the only available supply, and special treatment is deemed necessary. The supply can be drawn off by weirs and by-cuts leading to reservoirs, whence it will probably have to be pumped to the storage reservoir or service reservoir as the case may be, where there will be special apparatus for filtering, purifying, and otherwise dealing with the supply. For irrigation purposes, however river water stands paramount.

In flat ground near the sea, the water would be obtained from rivers by means of shallow wells sunk in the ground near the banks. They depend on continual percolation through the river banks. Near the sea the tide would naturally have some effect on the yield by flattening the saturation slope. The water obtained by this means is usually of fairly good quality, but is quite liable to surface pollution; in fact, any shallow well has the same disadvantages, particularly in dry seasons, or where the wells have been subject to excessive pumping. This, by lowering the saturation surface, forms what is adequately called by Messrs Tudsbury and Brightmore in their treatise on Waterworks "a cone of depression," causes a sort of suction on the upper surface, and is liable to introduce polluted water. But when well pass into ground below the saturation surface we have what are called deep wells, and when their formation causes the water to rise to the surface we have an artesian well, and the rush of water from such wells and boreholes is sometimes very considerable. Such borings and well-sinking demands, of course most particular attention to the place where the work is to be executed. The geographical and mineralogical conditions of the substrata are the all important considerations. Now such deep wells will usually pass through an impervious band before arriving at a pervious strata, but where such is not the case we shall have the same risk of pollution by excessive pumping as the shallow wells if a cone of depression occurs. Regarding rural wells, principally for small private supplies only, the water is frequently obtained by digging into the ground at a convenient spot and to the required depth, such depth being very variable, and depending entirely on the geological formation of the surrounding country. It needs only a little personal inspection to become thoroughly satisfied that the manner in which the majority of wells are constructed is not satisfactory.

The position of the well is usually decided, first, by its convenience to the dwelling-house, and in many cases, personally visited, the well was found to be placed within 6 ft. to 10 ft. from such dwelling-house. Often pigsties were adjoining; at least in one or two instances the pump was fastened to the pigsty wall, while the cesspool or "meg tank" was not more than 20 yds distant.

Dangers of Well Water.

The impurities from the soil, from defective drains and cesspools, readily gain access to the well and foul the purer water. It is therefore not surprising to find that the majority of the wells to farmhouses and in villages yield water which—to make a plain statement—is always liable at any time to become the means of spreading disease.

It is astounding to listen to the ignorant speeches and to witness the apathy of some farmers, especially should the water from their well be abundant in

quantity. To them it is an excellent supply, quality being a secondary consideration.

These farmers, however, should they be dairy farmers, are occasionally alarmed by the visit of the sanitary officer or the medical officer of health from a neighbouring town. Then, when the question of a fresh-water supply is suggested, and afterwards demanded, these farmers, although indifferent and scornful at first, often prove loudest in their praises when a new and perfectly pure water supply has been obtained, thanks to the skill and energy of the sanitary engineer.

The water supply to many farms and villages is from shallow wells, and it has already been explained how the water in these ordinary surface wells is readily polluted; at least it is a difficult matter to sink surface wells in connection with farmhouses, or the houses in many villages, without them becoming polluted. It therefore becomes necessary to sink wells of a greater depth, this depth depending on the geological formation of the ground.

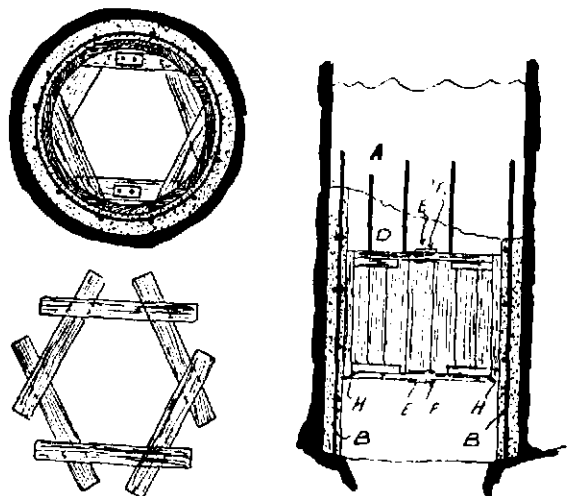
The quality of water yielded by deep wells is usually good, having some of the good qualities of spring water. It is liable, however, to be hard.

To decide on the proper place to sink a well is not an easy matter, and naturally this position must be correctly determined before any work is commenced, owing to the expense of such work. We cannot hope to place before the reader all the minor considerations which have to be gone into, and the services of a water expert are usually requisitioned before any actual engineering work is put in hand. No doubt the most favourable position for a well is a large synclinal (viz. dipping in opposite directions towards a common line or plane) basin having a good rainfall, the outlying strata being pervious, while the underlying is impervious. Where we have intersecting dykes of igneous rock outcropping, if a boring is made on the side nearest the outcrop of the strata intercepted, we shall be fairly likely to produce a good yield of water. There is no certainty about any boring; we can only be guided by previous experience. Wells and boreholes sunk in oolitic strata seem to have been the most successful, one at Bourne in Lincolnshire being one of the most remarkable, yielding (at only 92 ft. deep) half a million gallons per day at the surface, and borings in this class of rock need only be small; but when we come to sandstone and other similar rocks, we have to make them larger, because of the slower rate of percolation. Even then it is well known that where pumping is resorted to the yield will gradually fall off for a certain time after it has been in use, and it is most essential to avoid too much pumping where possible. Chalk is perhaps the most successful of all water-bearing formations, doubtless due to the high absorptive power of chalk, which may rise to as much as 2 gallons of water per cubic ft. (nearly $\frac{1}{2}$ of the whole volume). Being highly retentive, however, the yield, even if very dependent, will be of necessity slow; in fact some are of the opinion that water will not percolate direct from chalk, but only by the natural fissures which it contains, and the writer is inclined to concur with those views. An outcrop of chalk is better than a mere bed. Although oolitic rocks are very good, they are at the same time particularly liable to pollution. Now with deep wells, artificial lining, if they are in soft earth, is absolutely essential. If of brickwork, the process is the same as explained for tunnel shafts, the brickwork being very carefully set in cement and pointed. Cast-iron cylinders can also be used if the ground is very bad. The process of lining a well is called "steining" it. Besides preventing the collapse of the sides, steining has the advantage of preventing the ingress of polluted surface water, and is to be advised in all cases

of wells sunk through impermeable strata. The process of steining wells with concrete seems now to be a prevalent practice; and provided the best materials and workmanship are used, is very satisfactory.

* "Reinforced concrete is a very suitable material for steining a well, and has several advantages over brick or stone. Concrete, being impervious to water, completely shuts out the top surface-water, which is very liable to be contaminated with sewage or organic matter. In the accompanying illustrations, fig. 746 is a sectional elevation of the well, showing the circular form and reinforcement in position; fig. 747, method of cutting out the rings for the form; and fig. 748 is a sectional plan.

"The concrete wall is 5 in. thick throughout the height, reinforced with vertical steel rods A $\frac{3}{4}$ in. in diameter, spaced 1 ft. 6 in. apart, and for the circumference reinforcing rods B $\frac{1}{2}$ in. in diameter, spaced 9 in. apart. Patent



Figs. 746 to 748.

indented steel may be used instead of smooth round bars; these rods are square, the two smallest sizes being $\frac{1}{4}$ in. and $\frac{1}{2}$ in.

"The concrete should consist of 1 part of Portland cement, 2½ parts of sand, and 5 parts of broken stone or gravel. The stone or gravel should be well graded, varying in size from that which will pass through a 1-in. mesh and be retained on a $\frac{1}{2}$ -in. mesh. The Portland cement should comply with the British Engineering Standards specification.

"A collapsible circular frame 10 in. smaller than the diameter of the well and 4 ft. long is made and lowered to the bottom and adjusted in such a way that the concrete can be placed between the frame and the side of the well. The frame is constructed with two rings D, with an outer circumference of 1 ft. 2 in. smaller than the well; each ring is in two parts, which are joined by the two iron plates EE with bolts and nuts. In order to allow the frame to collapse, a taper piece of the ring is cut away at FF between the plates, and four boards forming the outer circumference of the frame at the joints of the rings are left loose. The simplest way to make the rings is to

• *Building World.*

draw a circle of the size of the frame, lay boards round its circumference as shown in fig. 747, then lightly tack these boards together, and, using the same radius, mark a circle on their ends as shown. The boards are then knocked apart and sawn out along the lines marked, and fastened together as shown. At FF tapered pieces of the ring 6 in. wide at the inner circle and 5½ in. at the outer circle are sawn out. Round the circumference are fastened the boards HH, which are 2 in. thick and 3 ft. long. The top of the rock is levelled off to make the foundation for the concrete lining, and as the depth of the ledge is 6 in. and the concrete wall 5 in., the circular frame will rest on the edge of the rock so as to mould the first section.

"The vertical $\frac{3}{4}$ -in. reinforcing rods are arranged evenly round the well 1 ft. 6 in. apart, and in the centre of the concrete. The horizontal $\frac{1}{4}$ -in. hoop rods are fastened to the inside of the vertical rods with wire ties, spaced 9 in. apart. In order to keep the vertical rods in position during the filling in of the concrete, their top ends are tied to a hoop fastened to the side of the well in the right position. The layers of horizontal rods are inserted at intervals of 3 ft. as the frame is filled.

"The frame is adjusted in position on the ledge of the rock, and in order to prevent the concrete sticking to the frame, the surface is covered with soft-soap solution, which is applied each time that the frame is raised. Concrete is placed a little at a time between the frame and the side of the well, and is well rammed with a small rammer so as to consolidate the concrete and thoroughly embed the rods. After filling with concrete to the top of the frame and allowing it to set overnight, or until the concrete will bear the pressure of the thumb, it is raised 2 ft. 6 in., braced securely, and the operation repeated until the ground-level is reached. In order to prevent surface water draining into the well, a slab of rough concrete 4 in. thick and from 7 ft. to 9 ft. in diameter should be placed round the top of the well on a foundation of broken stone, well rammed, 8 in. to 12 in. thick. In order that the wall of the well may be solid without voids, the concrete should be mixed rather wet."

Now, in addition to wells, we can obtain water from what are termed boreholes. These are really wells of a very small diameter, and instead of a man excavating the material inside and the well being steined, a special boring tool is used which bores a hole of small diameter gradually, the tool being lengthened, piece by piece, by special lengthening bars. These boreholes are not usually large, but holes up to 15 ft. in diameter have been formed by this process; but a smaller bore, called a *pilot* hole, is usually made first, say 5 ft. in diameter. The tools used are of special construction, and it would be too long a process to fully describe them here, but they are usually steam-driven, both a rotary and dropping motion being imparted. Boreholes are usually about 6 in. to 12 in. in size, and may be sunk to great depths, 1200 ft. not being by any means uncommon, but at such depths, in order to facilitate the working, the latter part is of a somewhat smaller diameter. They may be lined or not as required, with a cast-iron shell, or solid drawn copper tubes. The process of boring small holes is usually confined to rock; for those of larger diameter a shell pump is used to raise the debris. Then there is the diamond drill, which has special qualities. It consists of a core tube of wrought iron or steel, to which the crown is screwed, the whole being connected to the surface by hollow rods. It also has, in the later forms, the advantage of extracting a hollow core, practically intact. This has proved of service to engineers who wish to examine the nature of the strata bored through. Of course the tackle for this class of work is necessarily very

heavy, and shear legs, etc., have to be rigged up over the site before any work can be started. At the same time diamond drills of 3 in. in diameter and similar sizes are frequently used.

The necessary rotation is usually given by a rope running round a steam windlass, while the up-and-down motion is imparted by a special steam piston. The waste material is removed by a shell pump in the usual way. This is a simple apparatus, consisting as it does of a wrought-iron tube having a rubber foot-valve. By being raised and lowered into the rubbish a couple of times it gets filled very much in the same manner as the ordinary lift pump. It is illustrated in fig. 749. In soft ground, of course, a lining is essential. As stated, it may be of steel or copper tubes, or cast iron if desired; the

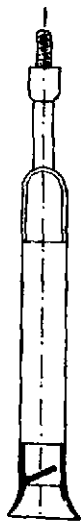


FIG. 749.

lowermost tube should have a cutting edge. The joints are usually screwed, and sometimes the practice of boring holes in the tube and forcing cement grout into the space between the tube and hole lining is resorted to, especially if there is much water getting into the bore. It certainly makes a better job, but the holes made in the tube must be very carefully plugged. This is done by drilling and tapping them, the plugs being screwed in. At the same time the efficiency of the procedure may be regarded as somewhat doubtful. It must not be supposed, however, that every well so made will yield a copious supply of water at the surface; such is by no means always the case, and special pumps have to be provided for raising the water to the surface, as explained under the heading of Pumping Machinery. Another way of obtaining a good supply of water is by means of what are technically termed *filter galleries*, by means of which a supply is satisfactorily gained from lakes and rivers.

The process consists of driving a series of tunnels into the water-bearing strata, the water from the river or lake filtering in to the galleries, from which it would doubtless be raised by mechanical power.

The walls would be masonry, brickwork, or concrete, and likewise the covering arch, but the floor would consist of about 12 in. of coarse gravel, the top of the arch being made practically on the same level as the dry-weather flow in the river.

But it would not be absolutely necessary to build such galleries to drain off the river water. The surface and sub-surface water naturally finds its way eventually to the river, and such galleries may be made to intercept the flow before it got to the river and so form a supply; at the same time, if the galleries were any way near the river itself, excessive pumping might cause the first-named action to take place instead.

Available Water Supply.—We have now to consider how we are going to arrive at the quantity of water available in any waterworks scheme. We have already touched upon the rainfall, and stated that we cannot in waterworks schemes depend upon the average dry-weather flow, but must depend only on the minimum, at the same time deducting $\frac{1}{4}$ at least for loss by evaporation. The duty of a storage reservoir is to equalise the flow over the longest known drought, or two or three smaller ones, which makes the study of good local rainfall records essential.

Again, we have to consider the facts that light rains will practically be all absorbed by the ground and yield nothing to the water supply. The number

of days' storage will therefore vary. According to the intensity and duration of the natural rainfall, 120–240 may be considered as the outside limits. Take the following instance. An average rainfall of 36 inches, deducting $\frac{1}{4}$ to arrive at the minimum, we have 30 in. available. Again we shall suppose $\frac{1}{4}$ (which is the least we could safely consider) to be loss by evaporation, leaving 20 in. for the supply to the reservoir. It is supposed to extend over the whole area, although, as we stated, over a very large area this may not be the case. Taking 1000 acres, we have
$$\frac{1000 \times 43560 \times 1.66}{365} = 199,200 \text{ cubic feet}$$

per day, of which probably anything up to $\frac{1}{4}$ may have to pass down the stream as compensation water to millers and other users of the stream. At the same time we are taking large figures, and it would be hardly fair to suppose that more than $\frac{2}{3}$ of this quantity would be available in dry weather, although it would be greatly exceeded in flood times. Actual observations show an average dry-weather flow in streams of almost 30 cub. ft. per minute off 1000 acres. We can then decide that 120 days' storage would be sufficient in places in which the rainfall is both frequent and more or less severe, but in places where it is light and infrequent it would be prudent to cause at least 240 days to be calculated for. Again, it is not absolutely necessary to base reservoir calculations upon rainfall statistics. Where records of stream gaugings date any way far back, they are much more reliable, because we know (or should know if they are properly done) the actual available quantity without making any assumptions, which are always inadvisable in engineering practice. However, any rules of the foregoing description can only be regarded as giving approximate results, owing to the nature of the ground upon which the reservoir stands.

Quantity of Water consumed.—This is the next thing to consider when we know the available quantity, because we want to construct our storage reservoir so that the minimum available quantity will be able to supply the maximum demand throughout any period of time. We must be able to provide for an adequate supply, but that is not all; sufficient pressure must at all times be available to force the supply to the tops of any of the town's houses. This, of course, will be governed to a great extent by the flow in the pipes and friction thereof. We have discussed the matter under Hydraulics. In the same way as we pointed out in dealing with a sewerage scheme, we shall have to arrive at the probable increase of population, because although we do not want pipes and mains of larger size than necessary, we do not want to be taking them up as inadequate in a few years' time. A little judgment is necessary. Obviously the flow will vary. The hot summer days will cause in big cities an increase of 20 per cent. at least, and in such places as seaside resorts, with a summer population, the rise will be much greater, while hard frosts will have the same effect owing to the large waste due to burst pipes, and people who would leave their water running continuously rather than avoid the risk of such bursts.

Again, consider what a sudden outbreak of fire would do. In most districts 25 gallons per head per day would be considered a fair average, but where there is likely to be much loss by waste, which of course must be avoided as much as possible, the quantity will be more, 30 gallons per head per day being, in the writer's opinion, a very fair average. These figures are, however, subject to considerable variation. In America, strange to say, the water consumption is far and away ahead of that in Britain, sometimes reaching 120 gallons per head per day.

CHAPTER XXXIII.

RESERVOIR CONSTRUCTION.

PRELIMINARY SURVEYS.

THE remarks already made in reference to Surveys should have made the reader fairly conversant with the principles of a survey for a waterworks scheme; at the same time there are certain considerations with special reference to this class of work which we will place before him. Like any other public works, there is the parliamentary and working survey.

The first thing to do is to range out the centre line of the proposed embankment. When this is done we must take the cross sections in the usual manner, and these must be parallel to the centre line of embankment. They are usually taken at 1 chain apart, but this rule is not binding. The

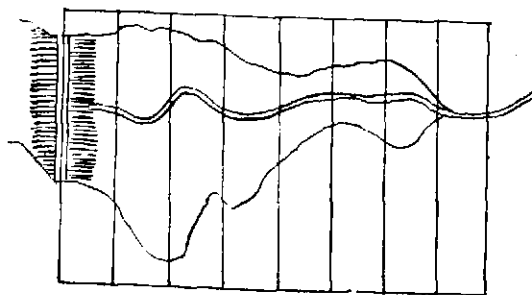


FIG. 750.

centre line of the reservoir would be along the line of the stream therein, and this is generally of a tortuous nature.

The work of laying off the lines of cross section from this would be a very tedious procedure, to avoid which most surveyors range out a line on each side of the reservoir, somewhere above the proposed top water-level, parallel to each

other and at right angles to the centre lines of the embankment, at right angles to which, and parallel to the embankment, the cross sections are easily ranged in. The procedure is illustrated by a diagram in fig. 750. It will then be necessary to take cross sections along the embankment at right angles to the centre line of that embankment.

When these cross sections are plotted, contours can also be laid upon the plan. These will be a guide in arriving at the approximate line of top water-level, whence a correct plan of the actual reservoir is easily drawn. It is also the duty of the surveyor, when he knows the top level of the embankment, to lay off on to the plan the proposed line of the toe of the bank and other details.

The line of the by-wash and its sill-level above O.D. will require to be marked on the plan also, together with fences, hedges gates, existing and

proposed roads, buildings and other features, as is usual in such works. When the reservoir plans are all finished he will have to lay off, by means of Ordnance maps or other ways, the proposed line of pipes or conduits, and carefully level the same. Of course the construction of an open or closed conduit *not* under pressure will be a matter of some care and consideration, but the line of a pipe *under* pressure will not be so important, because the main consideration will be direct route; at the same time, summits in the line should be always avoided where possible, and as uniform a fall as possible be provided. They usually follow as nearly as possible the line of the public roads, because of ease of repair and transit of the pipes, especially when large, 2-6 to 3-6 in. cover being usually deemed sufficient. The section should be taken along the invert of the pipes, being frequently referred to Ordnance datum. Pegs must be driven in the usual way along all lines of cross section.

STORAGE RESERVOIRS.

Reservoirs which collect and store the natural supply of water as it comes from the surface are called storage or impounding reservoirs, in distinction from service reservoirs which are more or less to regulate the hourly fluctuation of popular demand rather than the regulation of the supply over the dry seasons. The storage reservoir also usually supplies the compensation water to streams. We have already touched upon their necessary capacity, which is governed by the natural yield and the probable consumption, and it fulfils the following functions:—

1. Preventing draughts.
2. Preventing floods to low-lying country.
3. Increasing when necessary the flow of the stream which supplies it in the lower reaches.
4. Storing up the excessive floods which would do damage.
5. Allowing the mechanical impurities to settle to a certain degree.

As before stated, in order to ascertain accurately the required reservoir room, which should be such that ample supply is obtained, at the same time with no excessive expenditure:—

The site of the impounding reservoir is an all-important question. It must be at a sufficient elevation to provide good pressure at the consumer's end, and this is not always easily obtained. At the same time, the greater the distance from the consumer the greater the expense.

The whole of the site must be upon impermeable substrata, and this must be carefully ascertained by the engineer by means of trial pits and borings, and in the same way most particularly must the site of the embankment be so examined; no amount of time or money should be spared in these all-important details. Compact rock will, of course, be as good as any; in the same way clay, provided its thickness is ample; but fissured rocks, outcropping beds, etc., must be carefully considered, and the bad places excavated and filled with impermeable material. Provided clay is present in fair *quality* and *quantity*, it supplies in itself a suitable material for the embankment. When this is not the case, masonry and concrete have to be resorted to. It is usual to choose as a site for a storage reservoir a natural depression through which a river flows to supply the water, besides the other tributaries which are led to the reservoir from all parts of the catchment area, together with springs and subsurface waters which may find its way therein by filtration, materially assisted in many cases by a system of land drainage.

Earthen banks are usually of trapezoidal section. The thickness at the top requires consideration. Four feet for the puddle wall may be taken as a minimum, but the total top width of the bank may be much more, especially if a roadway is to be constructed thereon, as is very often the case. The excavation for the trench must be started and carried down to impervious substrata. When this has been done in accordance with the usual principles of such works, and the sides timbered up where necessary, the engineer must very carefully examine both the sides and the bottom for faults. Any such faults must be cut out and filled in with puddle before any part of the wall is commenced. It is also a mistake to step the bottom in longitudinal cross section too much, because of the risk of unequal settlement resulting. Sometimes great depths have to be sunk in order to attain a firm and impervious bottom. It is usual in firm ground to sink the puddle trench with sloping sides, and so avoid any timbering, as this does not appear to materially weaken the wall, otherwise a vertical trench is sunk. The withdrawing of the timbers will present some difficulty too, and demands much skill, so as not to disturb the puddle.

There are two kinds of puddle used, clay puddle and gravel puddle, the latter having small stones in its composition, and generally looked upon with favour for reservoir works. The stones, however, must not be angular. The puddle to be used must be thoroughly weathered, technically termed "souring" the clay by exposing it to rain, sun, and frost. When this has been properly done and gravel incorporated with it if required, it is laid in layers on the bottom of the trench and worked by long-bladed spades specially for the purpose, backwards and forwards, and by cutting and cross cutting, so as to thoroughly incorporate it with the layer below.

The work here mentioned must be done with the utmost possible care, as on its success the whole future of the works depend. Water is added as is necessary for working the clay, and as uniformly as possible, until the clay is quite plastic. We will assume that the work has been carried up to ground-level, when the process is slightly different. The wall will gradually taper upwards to the top, usually at the rate of 1 in. per foot on each side. This consideration and the width of the base will determine the width at ground-level. It is also essential to carry the bank well into the hillsides at each side of the reservoir to prevent leakage round the edges, and it is hardly necessary to add that the narrowest neck of the natural depression is selected for the puddle wall, provided all other conditions favour the choice. From ground-level upwards the earthen embankment is tipped in layers of 3 to 4 ft. thick as the puddle wall proceeds, because it is essential that the moisture in the puddle wall shall not dry out; cracks in the clay are very hard to remedy satisfactorily. From end to end the engineer should aim at raising the embankment as simultaneously as possible.

The inner part of the earthen embankment is always made of selected material, which should be carefully picked out of a finer grain and texture than the rest, and the other material reserved for the outer layers. The finished slopes demand attention. The inner slope is generally made with a batter of 3-1 and the outer one 2-1, which usually will satisfy all requirements. The inner slope is usually hand-pitched with dry rubble, and the outer one soiled and grass seeded. The above reasoning is shown illustrated in practical form in figs. 751 and 752. The top width is usually 20 to 30 ft. according to requirements, and should be about 10 ft. (never less than 6) above top water-level, to cope with the waves likely to rise up, which they do, and sometimes

with considerable force, in exposed situations. Settlement will then take place and will probably continue uniformly for some considerable time; until the engineer has observed its termination, the hand-pitching referred to should not be attempted. During the construction of the reservoir two things must also receive attention at the same time as the puddle bank—that is, the by-wash and the outlet culvert, and also the waste weir. The by-wash is a special cut or canal laid along the hillside to take away the water during the progress of the works. Its level is usually above the top level of the bank where possible. Then we have the outlet culvert. This may be constructed

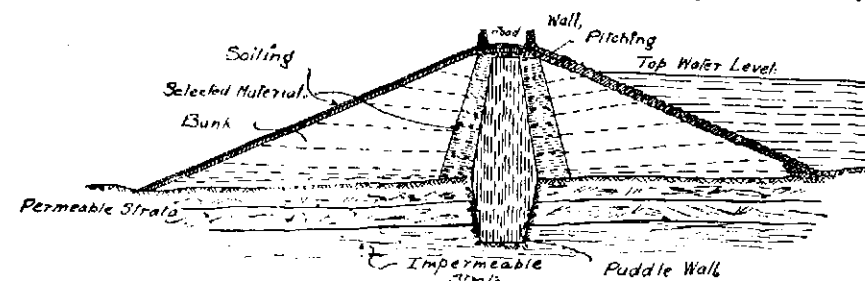


FIG. 751.

in the actual puddle bank, or it may tail into the bank round the ends of the puddle wall. The reason for this obviously expensive construction is that less disturbance is likely to occur to the puddle wall, and hence less likelihood of failure and leakage, and also the continual settlement of the puddle is at all times likely to cause fracture to the masonry or brickwork culvert, and it is the opinion of many engineers that it should be carried round the ends of the

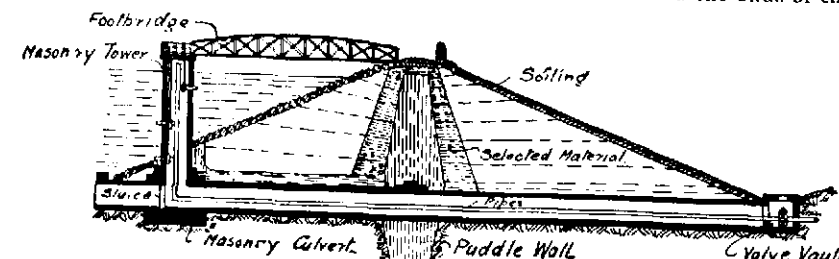


FIG. 752.

wall. It is in this case that the by-wash is required, but when the outlet culvert is in the middle of the bank it is more usual to construct it once for all and allow all water to flow through it. At the same time, the centre of such a culvert should be supported not on the mere puddle, but a solid wall of masonry or concrete. The use of slip joints on the culvert is not to be advised.

The shape of such culverts is sometimes circular; theoretically, an elliptic section of ratio of horizontal to vertical axis of 2 to 3 is best, the bottom being usually a short inverted arch. They have been, however, constructed of all sizes and shapes. They must in all cases be very carefully backed with puddle, or preferably Portland cement concrete. They are widened out at each

end into two bays with wing walls, called the "fore bay" and "tail bay" respectively, according as the inner or outer end is referred to. And if we have d = diameter of culvert when circular, and the smaller diameter when elliptical, and H = height of earth above it in feet, we can find the correct thickness of the masonry by the following equation, the result being given in inches, d being in feet,

$$t = d \sqrt{\frac{H}{2}} \quad \dots \quad (402)$$

A typical culvert section is shown in fig. 753. The pipes are then all laid in this culvert, and it should be of ample size for repairing them too, and either a valve tower is provided or a masonry diaphragm (or "stopping" built into the culvert when it is desired to fill the reservoir, the former being the usual modern practice. It should be borne in mind that the fore bay and tail bay are not merely structures, as they may at first seem, for ornamental purposes only, but they have to withstand a large thrust of the embankment

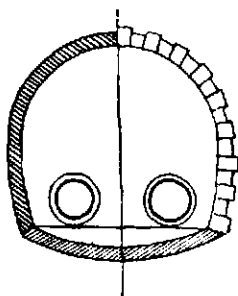


FIG. 753.

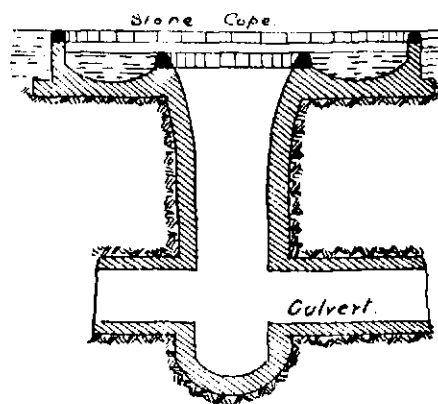


FIG. 754.

tending to spread, and hence their foundations demand attention. They must be treated as surcharged retaining-walls. The waste weir is likewise an important part of the whole works of a reservoir. It must be of such size and construction that the water in the reservoir shall never rise to a dangerous height, and we gave formulæ for its size under "Hydraulics." Naturally the depth on the crest together with the maximum discharge fix its dimensions. The former quantity is not generally allowed to exceed 2 ft. in practice, while, in addition to what we said in the previous chapter referred to, if we know the maximum flood discharge likely to be encountered in cubic feet per second and divide the quantity by 8, we shall have a very good rule for arriving at the safe length of the waste weir.

The actual construction would be the same as any other masonry weir and we would refer the reader to the chapter on Weirs. That is to say, heavy masonry should be employed, and the water not allowed to descend too fast but rather be broken up into a series of falls, but each fall should in no case be more than 2 ft. vertically. An ordinary straight weir is desirable, as far as possible at right angles to the flow, while circular waste weirs of the form shown by fig. 754 have met with some success. Another form of reservoir

construction is shown by fig. 755, in which we have a concrete-lined reservoir, which might be a common occurrence under the circumstances.

The chief difficulty will occur at the junction of the rock with the filling above ground level, and there at least it will be desirable to have some reinforcement, say expanded metal No. 15, $\frac{1}{2}$ -in. by $\frac{1}{2}$ -in. strands, 3-in. mesh, 4 lbs. per sq. yd., in pieces 6 ft. long, up and down the bank, and whatever width they run. If there is a probability of the filling yielding, from want of sufficient time to consolidate, it may be desirable to put a layer of expanded metal in the centre of the thickness over the whole surface of the filling. This would also be a safeguard against cracking from unequal expansion and contraction.

In some cases it may be deemed necessary to provide for a concrete or masonry dam of large section, preferably to a puddle one. A puddle wall needs no calculations as regards strength, providing ample top width and the usual batter to the sides is provided. But dams of masonry demand very accurate and sometimes intricate and laborious calculations, because we must provide against risk of failure; at the same time, we must not waste a more or less expensive material. We treated at some length the subject of Retaining-Walls in a special chapter, and also gave some rules whereby reser-

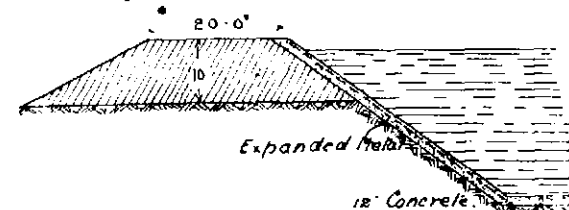


FIG. 755.

voir walls of considerable height could be designed. Both the methods stated would give good results without a large amount of labour. These dams also form a very convenient waste weir along their whole length, and are usually constructed to provide this. They are often of huge dimensions, anything over 100 ft. high, this being considered as the safe limit for an earthen embankment. Blocks of masonry are used, the larger the better, provided they are sound. They may be uncoursed, but should be squared on the exposed faces, every small space being very carefully grouted with concrete, a process upon which the success of the work primarily depends. It may be curved, as we stated in the chapter referred to. Besides the fact of the water flowing over the weir in a dam of this description, which it may be remarked should have a rock foundation, sluices may be provided on the dam itself, with the object of letting off sudden floods. Such a design of sluice for working under a considerable hydrostatic pressure is shown in fig. 756, of the type manufactured by Messrs Ransomes & Rapier, Ipswich, who kindly supplied me with the photograph.

This photograph represents a sluice of the submerged type, it being one of the smaller sluices made for the Assouan dam. The size is 11 ft. 6 ins. by 6 ft. 7 ins., and the head to sill, when the heightening of the dam is completed, will be 56 ft.

The sluice is shown erected at the works in its frame, and lifted to half its full opening.

The gate is constructed of mild steel, and the frame of cast iron.
One hundred and eighty sluices of this type were made for the Assoua dam, 60 being of the size shown on the photo, and 120 of 23 ft. deep by 6 f

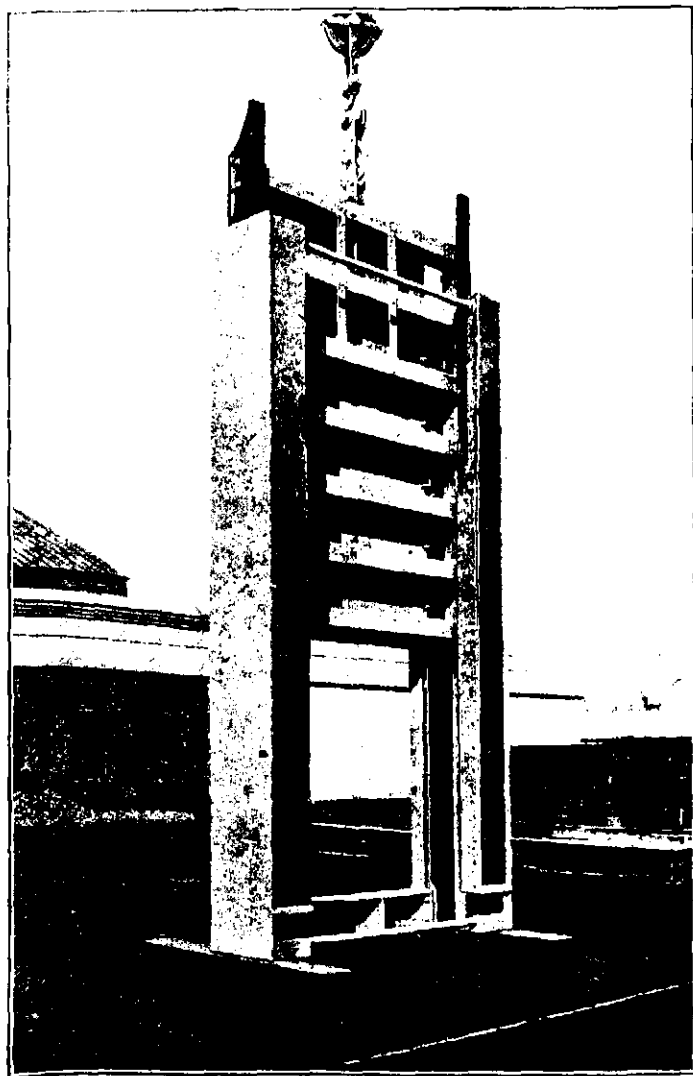


FIG. 756.

7 ins. wide. The maximum head to sill-level on the lowest of the deep sluices will be 84 ft. These sluices are not counterbalanced, and are lift by means of steel wire ropes from crab winches fixed at the top of the dam.

The subject of the several stresses set up in very high reservoir walls of a somewhat complex nature. It would be too long to discuss here, but

would refer the reader for a very good account of the subject to *Water-works Engineering*, 3rd edition, by Tudsbery and Brightmore, chapter iv., and Rankine's *Civil Engineering*, p. 780 *et seq.*, and the *Engineer*, January 5, 1872, for further information; at the same time the reasoning of the previous chapter mentioned should give the reader a fair idea of what calculations are necessary.

Before going further with our subject, it would be well to briefly state what we mean by compensation water. It is a matter of difficulty to divert and impound water and not affect anybody's previous rights; where the water is impounded and millowners and other parties downstream suffer thereby, compensation of some sort becomes imperative; and unless such agreements are come to, Government will not sanction such works to proceed. The affected parties are usually naturally very much opposed to such schemes, sometimes with success, and cannot always be convinced of advantages which the works are likely to offer them. We are not concerned with monetary compensation, that is more a matter of law, and it is water that we are principally concerned with. It is here that the form of leap weir mentioned in connection with sewerage works comes in useful, because the sudden flood waters are well diverted from the reservoir owing to their turbid nature, while they will do well for the purposes of the millowners, and such separation of flow is usually provided for by storage reservoirs. Now it has been the practice of past years to give up $\frac{1}{3}$ of the available yield to millowners, but that does not say that in times of dire necessity that quantity could not be turned into the reservoir; at the same time the parties concerned would look for, and probably obtain, monetary compensation. But where a special storage reservoir is provided for the purpose of compensation water only, the millowners will mutually benefit thereby, because they will have a regular flow; floods which hitherto provided no advantage to them will be stored up, only to be doled out to them at times when the mills would be stopped owing to a prolonged drought, because nowadays the modern turbine is replacing the old-fashioned water-wheel, and the extra head due to a flood would not have very much effect upon the output. Again, the night flow, which hitherto went to waste, is now turned to good use. It then appears that perhaps the advantages accruing from a large store reservoir for power purposes as well as domestic supply would be such an advantage as to induce the millowners to pay towards the construction of the same. According to Mr Slagg in his *Water Engineering*, p. 133, 2nd edition, a horse-power would be worth about £12 per year, and 12 cubic ft. of water falling 1 ft. per second may be considered as equal to 1 horse-power. Therefore if an outlay say of £20,000 was made on a reservoir which would directly provide 85 horse-power only, we should have a return of 5 per cent. per annum on the outlay.

Regarding the appurtenances of reservoirs, we have already mentioned the by-wash leap weir, outlet culvert, and waste weir. Besides this there will usually be provided a fish pass and the all-important valve towers. The fish pass is a special contrivance. In ordinary cases of low weirs on rivers and canals the old salmon stair or ladder answered all requirements, but with the high embankments constructed nowadays a special contrivance is necessary, and the most successful form yet devised is known as MacDonald's fish pass. Regarding valve towers, the original practice was to make them of masonry, circular in shape. Some of these towers were very ornamental, and reflected credit upon their designers, and continue to give satisfaction. They are a simple and straightforward piece of work, provided the masonry is designed

mostly confined to drawing off the clearer liquid from sewage tanks. Yet another way is by means of the syphon; and although not much practised, it certainly does away with all risk of interference with the puddle bank. The only consideration is that the level of the outlet must be below that of the inlet such an amount as will provide sufficient head to overcome the friction of the pipe line; 1 ft. will generally suffice. There must, however, be a means of charging the pipe when starting. If a supply of water at a height above the top of the syphon is handy, it can be easily filled by closing both valves (one at each end); but failing this (and it is not likely such a supply would be handy, for obvious reasons), an air-pump must be provided for the purpose. At the same time the judicious use of a hydraulic ram somewhere upstream might provide an adequate supply of water for the purpose. Another detail is shown in fig. 761A.

The next subject to be considered is that of service reservoirs. Their duty is not to regulate the supply over long periods of drought, but rather to equalise the daily and hourly flow, and provide storage in the case of breakdown to mains and engines. While the storage reservoir may be many miles away from the town it supplies, the service reservoir would be in the near vicinity. They must, however, be at a sufficient elevation to provide adequate pressure at all points of the town. Now, while a storage reservoir would naturally be, in the country, amidst more or less pure atmosphere, not so the service reservoir, which would be liable to pollution from the town atmosphere, and it is now the practice to cover these reservoirs in ways we are going to describe. It is usual to form such reservoirs half in excavation and half in bank. They are usually constructed of brickwork in cement, plastered inside with neat cement, or rendered waterproof by asphalt or other composition of a similar nature, while the roof generally consists of a series of arches, or if the work is done wholly in concrete, which is now the tendency, by a system of steel joists supported on cast-iron columns, and the intervening spaces being afterwards filled with concrete; or, again, a series of arches may be turned between the joists to form the roof. The subsoil on which they stand must be thoroughly well drained before the floor is put in, or else a frost may result in complete fracture of the whole affair. The walls also demand attention, because they will be subject to hydrostatic pressure from within, especially above the ground-level, and they are treated in the same way as any ordinary retaining-wall, and strengthened with buttresses if required. It is also the practice, where puddle is handy, to back the walls with a lining of the same, as an extra precaution against leakage. Inlet and outlet pipes are, of course, necessary, and also a scour pipe for emptying purposes, the floor having a gradual slope to this pipe. It is usual to place on the inlet pipe a reflux valve to prevent any oscillation of the water therein, while stop valves are of course necessary on both pipes, while a bell-mouth casting should be provided upon the outlet, for reasons stated in our chapter upon Hydraulics. A form of reflux valve suitable for the purpose is illustrated in fig. 762. Again, ample ventilation must be provided for the tank in the roof. These should be of a simple and inexpensive nature, because there is no necessity for any fancy design, types of which often occur in manufacturers' catalogues,

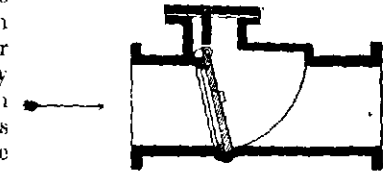


FIG. 762.

to withstand the greatest pressure likely to come upon it, and that the joints and stonework in general are of the very best quality. The minimum thickness would be about 2 ft., and the outer face would taper upwards. The foundation would have to be of a very stable character. Inside the valve tower are the draw-off pipes, down-pipes, valves, etc., and at the base is the scour pipe, while there are also screens of copper-wire gauze carried on frames. These must always be provided for in duplicate, and must be capable of easy raising and lowering. They are to rid the water of the larger suspended foreign matter. The screens are usually placed inside the actual tower itself. In some reservoirs, especially the smaller ones, it would not be absolutely necessary to have a valve tower; the pipes may be taken directly through the culvert, which has a masonry stopping. In this case bell-mouths must be provided on the inner ends of the pipes, and the valves there also. They would be worked from a small platform, and should always occur in duplicate. In no case whatsoever must a supply or any other pipe be laid from a reservoir having only a single valve at the outer end of the culvert. Again, it is not absolutely essential to make a culvert, but it is no doubt very advisable, owing to the enormous stresses likely to be set up in the pipes laid on the puddle trench, and, owing to the facts that cast iron is still such a very unreliable material, and that a failure due to fracture would be very difficult

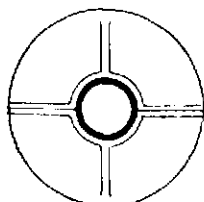


FIG. 757.

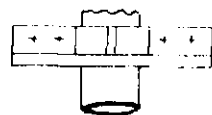


FIG. 758.

to rectify it is difficult to see why engineers have laid the pipes direct in the puddle bank. We do not advise the procedure in large reservoirs, but when so, the pipes must be special castings, and have what are termed puddle plates thereon, figs. 757 and 758, of cast iron. They will obviate the risks of unequal settlement, and also tend to avoid water creeping along their length, which would eventually lead perhaps to a serious leakage. The modern tendency is to make valve towers of cast-iron segments, as being more economical and obviously more adapted to the purpose. A good form of tower on the principle is that of the upper Cwenbran reservoir. It is shown illustrated by figs. 759, 760, and 761, and was made by Messrs Blakeborough & Co., engineers, Brighouse, Yorks, who kindly supplied me with the drawings in question, which are self-explanatory. All valve towers want a small foot-bridge for access. One of the objects of a valve tower is to enable the water to be drawn off at any level; that near the bottom, usually having more or less sediment, is not so good as that drawn off near the top. It is also a means of access for repair to the valves, etc., and for the screens. The scour pipe is usually contained in the culvert also, but in some places it is the practice (not a very good one, in the writer's opinion, although such a provision may be made for an emergency) to run the waste water directly into the culvert by means of a sluice. Valve towers should be painted white inside and have a glass roof, providing plenty of light. Another way of drawing water off from the top of a reservoir is by means of a floating arm. The practice is, however,

and only tend to add to the expense. The actual construction of a service reservoir has a few points worthy of notice:—

1. All pipes should be laid in loose holes in the walls. After final settlement of the structure has taken place, these holes must be very carefully plugged with cement concrete.

2. The lining of the reservoir, though simple perhaps to the uninitiated, demands great care, for on its efficiency the whole work primarily depends. It should be entrusted only to firms who make a speciality of this class of work.

3. Where possible, in the case of an outer puddle lining being deemed expedient, the footings of the foundations should be splayed out instead of stepped out. This prevents the liability of the puddle to crack. The excavated earth is afterwards thrown up over the top and sides, and sloped off in the usual way, and sowed with grass seed. This tends to keep the water cool during summer weather. Two to three days' storage in a service reservoir would be considered ample for most purposes; sometimes even less would be provided.

Again, the storage reservoir may be so near the town that it may be deemed less expensive to lay the mains in duplicate, and so reduce risks of failure of supply to a minimum. Another use to which such reservoirs may be put is that of supplying a local authority with water, under payment from a separate water board. It provides a means of storage, and more or less independence on the part of the local authority, while at the same time providing constant pressure, irrespective of other circumstances, and also a suitable place for measuring the supply so obtained by suitable weirs as well as meters. The quality supplied can also be kept under observation. In addition to such reservoirs, which may be considered to be of fair proportions (in fact, in places they attain huge proportions, for example the Honor Oak reservoir, opened in 1909 by the Lord Mayor of London, for the Metropolitan Water Board), we can have storage tanks for small districts of much smaller proportions, providing an artificial head by reason of their elevation. Steel was until quite recently the form of construction, and still holds its own even at the present day, as is seen by the huge steel tank recently erected in India, but ferro-concrete construction has come to stay, and it has a peculiar adaptability to such high service tanks, as shown in our chapter on Ferro-concrete by figs. 439 and 440. Again, the lining of such tanks demands the most careful attention.

A very favourite form of steel tank was that with the spherical bottom. The tank is circular in shape, and is supported on a circular tower of masonry or steel. The roof is pointed, and usually double, while in the centre is a shaft containing a spiral staircase for access to the tank. It may be lined with asphalt, but properly designed joints should meet all requirements (see Chapter XXXVIII.).

The means employed for filtering, purifying, and softening water have now to be considered. We say purifying, but in most cases the water does not undergo any chemical changes, and the operations are usually mechanical only, such as removing the suspended matter. At the storage reservoir are the screens, usually part of the valve tower, which has already been explained. These will rid the water of some of the larger particles in suspension, but they only provide a more or less preliminary treatment. The natural settlement of any water in the storage reservoir tends to cause a large amount of impurity to settle; that is why, for the purposes of supply, the bottom of a reservoir is never drawn off. Again, the flow of water along an

open channel, or a covered one for that matter, causes natural purification by aeration, certain gases, such as dissolved hydrogen sulphide, escaping thereby. Again, carbonate of lime, which is a very common impurity, will be deposited along the sides and bottom of the conduit, due to the evolution of CO_2 . Ferric oxide may be also deposited under the same conditions. Floating bodies would pass over the waste weir.

Considering the actual construction of the filter beds acting by gravity, a very usual form is shown by fig. 763. On the top should be a layer of coarse granular siliceous sand, about 2-6 deep, below which is gravel, 2-6 deep also, but laid in layers of different sizes as shown, that at the bottom being about the size of an ordinary egg. Along the bottom are laid special drains, which lead the water away to a clear-water tank by means of a central drain. It might be supposed that the use of settling-ponds would be all that is desired for clarifying a good supply of water. This may be the case, but the size of these tanks, consequent upon the slow process of subsidence, together with the fact that a long quiescent stay in such tanks (which must of necessity be shallow), will give rise to micro-organisms which would flourish

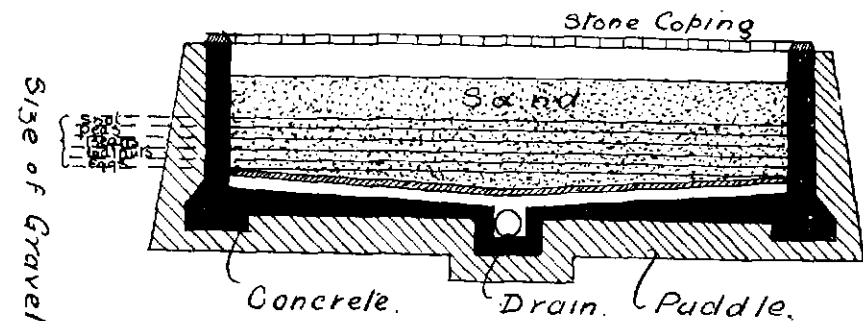


FIG. 763.

under those conditions. These settling-tanks may be of the intermittent type, allowing the water to settle and then drawing it all off, but the system has been found unsatisfactory, and given way to the continuous method, in the same way as sewage settlement. These tanks are built somewhat on the same principle as sewage tanks, that is, they have dividing walls, to cause the water to take a long tortuous passage through them. This causes the water which has been longest in the pond to be always drawn off. Such settling-tanks are usually made of concrete, and have sloping sides, usually at 45° , and the bottom is slightly sloped towards the scour valve. In the same way the walls of the filter beds and the bottom are made of concrete, having an outside lining of puddle, but the sides would be vertical, or have a slight batter, as the case may be. The remarks in reference to service reservoirs apply to them. Owing, however, to the small pressures set up in them, the walls are only of light construction, demanding careful construction to avoid cracking, so usual with these concrete walls. Continual wearing away of the sand will take place, which necessitates continual renewal. The inlet to such sand filters must be of special form. The idea is shown in fig. 764, and consists of a trumpet mouth-piece from which the water flows into a large shallow cast-iron dish to prevent any disturbance of the surface of the sand. The arrangement is that made by Messrs Blakeborough & Co.

Again, some form of regulating apparatus must be employed for filter beds. One of the latest patents for the purpose is Langford's automatic filter regulator. This apparatus consists of a galvanised iron float which carries and supports a three-legged syphon, the discharge leg of the syphon projecting downwards into the stand-pipe which leads the filtered water away to the clear-water reservoir. Thus, as the filter gradually becomes clogged, or the level of the water in the outlet pit changes for any other reason, the outlet end of syphon floats and is always kept the same fixed depth below the water surface. In other words, the head on the syphon never changes.

At the bottom of the discharge leg a specially shaped gun-metal regulating cock is provided, which may be opened or closed at will by means of the worm and worm-wheel acting through the central spindle.

The central spindle carries a graduated quadrant at its upper end, which, by its position in relation to the fixed pointer, indicates the degree of opening of the regulator cock, and so the rate at which water is passing through the apparatus.

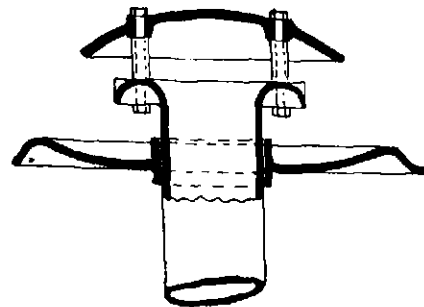


FIG. 64.

The index plate may be engraved to suit special requirements, showing either the total number of gallons filtered per hour, or per hour per acre, or the number of inches depth of water per hour. Any two of these scales may be conveniently engraved on the same plate.

Mode of Operation.—To charge the regulator, the larger sluice valve is closed and the small valve opened.

The air is withdrawn from the syphon through an air-cock at the summit by means of a small hand-pump. The small valve is then closed, and on the large valve being opened the syphon comes into action. When two or more regulators are situated near together, the air-pump can generally be dispensed with; all that is necessary is to connect the air-cock of the syphon to be charged with that of its operating neighbour by means of a $\frac{3}{8}$ in. rubber pipe. In this way the air is sucked out in a few minutes, according to the size of the apparatus; and this operation is only necessary after the water has for any reason been drained out of the filter bed. The apparatus is made by Messrs Blakelborough & Co. also.

Filters should deliver into a clear-water tank whose top level must be below the base of the filters in order to provide for the maximum filtering head. They must be covered, and more often than not provide in themselves the service reservoir as well. The drains of the filter bed demand attention, and are often of very special construction. It is essential to provide numerous drains, at the same time making them of large sectional area, so as not to waste any head in passing through them; enough head is usually lost in the actual filtering medium itself. Hollow bricks, laid dry, have been used with some success, while the collecting drain should be in the centre. Two forms of filter drain are shown by figs. 765 and 766, one being sunk into, and the other raised from the floor. Each end of the collecting drains of a filter require ventilators. Regarding the actual area of filtration, we stated before that 100 gallons in twenty-four hours per sq. ft. of filtering area was a suitable speed at which to work gravitation filters. This, however, may

be taken at the maximum rate of filtration, and it is usual to provide for filtering up to double this area, that is, for a rate of only 50 gallons per sq. ft. The sand should be not more than 2 ft. 6 in. deep. It has been proved that any extra depth provides no advantage, and for successful working the filters should be flooded to a depth of 2 ft. The apparatus for controlling this has already been explained. Each filter should be limited in area to about 3000 sq. yds. The whole effect of the actual filtering is really limited to the 2 or 3 top inches of the sand. When once started, this layer obtains upon itself a sort of jelly-like slime which really acts as the filtering medium, and is supposed, and no doubt rightly so, to have certain chemical effects upon the water which materially assist in purification, and for this reason filters are always started where possible with a flood of water from below. Naturally, after a time the sand gets in such a condition as to render the filter inefficient, and it has then to be cleaned. This consists in removing the top $\frac{1}{2}$ in. of sand and taking it to the cleansing dépôt, where it is washed by special machinery, after being well exposed to atmospheric influences. The remaining sand is then well ploughed and turned over and remade up to the proper level. Closed filters are to be

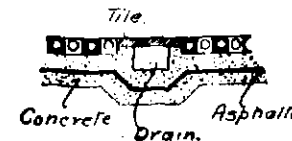


FIG. 765.



FIG. 766.

advised where it would be necessary to have the works near a town. It is true that they do not want such frequent cleansing, but at the same time their efficiency is impaired by the pillars which support the roof, breaking up the continuity of the medium.* The washing of the sand demands special attention and special machinery.

We now come to consider the mechanical filtration of water. This takes the form of either gravity or pressure filters.

In the *gravity filter* the process of filtration more nearly resembles that taking place naturally, and it cannot be forced beyond its normal rate of flow; but, generally speaking, and especially for industrial purposes, the scope of this type of filter is limited, and the *pressure type* is more applicable, since it can be worked under conditions unsuited to the gravity type.

In both classes the filtering medium consists of pure quartz crystals, crushed and graded to suitable sizes. These crystals are extremely hard and possess a peculiar sharpness, owing to which they are most effective for the purpose, and indestructible. In both types of filter, sterilisation of the quartz beds can be easily effected after washing out by forcing live steam through them, thus destroying all animal and vegetable germs.

The time required for cleansing the filters and the quantity of water used in the process are always small, but they will necessarily vary with the character of the unfiltered water, and the extent to which the filter is used.

The pressure available for cleansing is also important, as, with low pressure, washing out must be continued for a longer time than with high pressure.

* An engineering difficulty which should be capable of being surmounted by ferro-concrete.

Although the filters are primarily intended for the removal of solid matter in suspension in the water, it has been found, as shown by various reports that they also effect an important reduction in the bacteria and other organic impurities present, inasmuch as these are often in such close connection with the suspended solids that the removal of the one eliminates the other also.

Where the removal of either bacteria or colour is of first importance which cannot be effected by simple filtration with any ordinary type or arrangement of apparatus except with special care and at considerable cost, we can supply auxiliary mechanical appliances in connection with the filter proper whereby suitable chemical reagents, determined independently for each class of water to be purified, are added to the water before it enters the filters.

The apparatus in question is exceedingly simple, and can be attended to by an unskilled person; and no risk whatever attaches to its application even in the case of decidedly impure waters, where the highest degree of purification is necessary.

PRESSURE FILTER.

The patent pressure filter consists of a closed cylindrical tank with sloping sides and domed top and bottom, supported in the larger sizes on four legs. In the larger sizes the tank is constructed of riveted steel plates in the smaller it is of cast iron.

Inside, at the upper part of the tank, is fitted an annular channel through which the unfiltered water enters from the inlet valve, overflowing the edge on to the bed of filtering material, and passing down through it to the collecting chamber at the bottom. The filtering material consists of quartz crystals in graded layers, with the finest at the top; the bed rests on a dished iron plate, which separates the filtering from the collecting chamber. Into the dished iron plate a large number of brass nozzles are screwed from the under side; ready access to them can be obtained by means of a door fitted in the side of the collecting chamber, and they can thus be replaced without in any way disturbing the quartz bed. The object of the nozzles is to ensure the effective use of all parts of the filtering bed, as well as the proper distribution of the water used in washing out.

Besides serving to spread the unfiltered water over the bed, the annular channel also serves to carry off the dirty water when the filter is cleansed. For this operation the direction of flow is reversed; the unfiltered water inlet valve and the filtered water outlet valve are closed, the wash-out inlet valve and wash-out discharge valve being opened. The impurities filtered from the water collect, for the most part, in the top of the bed; and in order to facilitate their removal during the wash-out, a rake, which can be rotated by hand, is fitted inside the tank, so as to break up the surface of the bed during the cleansing process.

As soon as the water from the wash-out discharge valve is seen to be clean, this valve and the wash-out inlet valve are closed, and water again admitted through the unfiltered water inlet valve. This water, however, is not allowed to pass to the mains, the filtered water outlet valve being still kept shut, but instead, the re-wash valve at the bottom of the filter is opened for a few seconds to allow the first discharge to run to waste, so as to clear the nozzles and collecting chamber of any dirt which may have entered with the wash-out water, and to settle the upper layers of the quartz bed into a properly compact condition.

GRAVITY FILTER.

This type of filter is similar in its principal points of construction to the pressure filter already described; it has the same arrangement of dished plate, into which are screwed nozzles, separating the filtering from the collecting chamber; the filtering bed consists of quartz crystals, and can be stirred up by a rake.

The difference lies in the fact that the cylindrical tank is open at the top, and that the annular channel for the inflow of the water is fitted at a few inches above the surface of the quartz bed, forming a belt all round the filter, and communicating with the inside by a number of openings. As in the case of the pressure type, this annular channel also serves to carry off the wash-out water.

Messrs Mather & Platt, Salford Ironworks, Manchester, who are large makers of this class of filter, have kindly supplied me with drawings of the apparatus, which I am able to place before the reader. Figs. 767, 768, 769 show a filtration scheme consisting of a battery of ten of their patent pressure filters, each 8 ft. in diameter. The several pipes attached to the filters are joined in a common main in the pipe trench, and all valves are arranged to be operated from the ground-level. Therefore all that is required in the form of staging is a light gallery to gain access to the bearings for oiling purposes.

The water to be dealt with in the installation in question is of a peaty colour, and therefore it is necessary to add alumino-ferrie prior to filtration. Also to ensure that the water will be free from any plumbo solvency or action on lead pipes, whiting is injected into the main as well as the alumino-ferrie.

With the apparatus shown on this drawing the chemicals are added to the main in correct proportion to the flow, as a special form of water motor is supplied, the connecting rod of which is attached to a small measuring pump, which in turn supplies the chemicals to a small force pump.

As it is necessary to keep the whiting in a milky state during the whole period of filtering, an agitator is fitted in the tank, which is driven by belt from a small pelton wheel.

The agitating gear on the filters is driven by an oil engine, but space has been left for the installation of a pelton wheel, which it is proposed to install, so that two alternative forms of prime movers will be available according to circumstances.

We now come to purification of water by other than mechanical means, that is, by chemical processes. All waters, besides containing suspended matter which can be removed mechanically, also contain organic and mineral matters in solution which often require removal or neutralisation. The principal medium for this purification process is magnetic oxide of iron in the form of magnetic carbide, which is prepared from hæmatite iron ore. There is also another medium used, called spongy iron.

Ordinary gravitation filters of these materials, however, will rapidly clog, and special machines for their use, such as Anderson's purifier, have to be resorted to in order to keep the medium in a constant state of agitation. In most of the processes the effect of the iron is to convert the acids into neutral salts. These salts are soluble in water. Then, after passing through the purifiers, sufficient aeration must be given to the water, so that these ferrous salts will become oxidised. The result is that the CO_2 will escape, and the ferric oxide will, by virtue of its specific gravity, sink to the bottom of the settling-tanks and be capable of removal. There is no doubt that treatment

by iron will remove the organic impurities contained in water by suspension, and it will also successfully reduce nitrogenous matter and albuminoid ammonia. It is also stated that bacteria are destroyed in the process.

The hardness of water is a most important element in the supply, not perhaps so much from a domestic point of view as the manufacturers' and steam users'. From a steam-raising point of view, the worst impurities to deal with are the various bicarbonates and sulphates, and in sea-water common salt, because, on boiling, the bicarbonates will evolve CO_2 and leave calcium and magnesium carbonate. Calcium sulphate is the worst of them all, as it sets up in boilers a hard coherent scale. For peaty waters which contain organic salts, a little chalk added will prove an advantage. Most of the patent boiler compounds in use at the present day have an alkaline base, but before using any of them, both the composition of the compound and of the water itself must be carefully ascertained, because a solvent which would suit one water would of course be quite unsuitable for another perhaps. Lime on the Clarke system is a remedy for bicarbonate, or caustic soda would be an advantage under the same conditions; ordinary washing-soda would be sufficient for dealing with carbonates. There is a patent solvent on the market known as "tripea"; it is principally composed of tribasic phosphate of soda, and has the advantage of precipitating the salts of calcium and magnesium in a sort of slime at the bottom of the boiler, providing easy removal by the blow-off cock. The determination of hardness of water, in fact anything to do with the composition, is usually left in the hands of experienced chemists. Hardness is divided into two groups, temporary and permanent. The carbonates of lime and magnesia constitute temporary hardness. They are got rid of by the liberation of the CO_2 ; but calcium sulphate and magnesium sulphate, which are very difficult of removal, are termed permanent hardness. Besides the natural process of aeration, temporary hardness can be got rid of by boiling (hence the trouble in boilers) and the addition of milk of lime (Clarke's process). In settling ponds the water should be always drawn off by a floating arm.

Regarding the apparatus for sand washing, there are numerous designs to be seen in connection with waterworks schemes. One of the most successful pieces of apparatus which the author has yet seen is Everard's patent sand-washing apparatus.

This apparatus is especially suitable for washing sand for filtering water, the special arrangement, in circular form, placed within the beds, being designed by J. B. Everard, Esq., M.Inst.C.E., Leicester, for the Leicester Corporation's Swithland scheme. It comprises a series of hoppers, near the bottom of which a jet of water under high pressure issues from a nozzle in a horizontal direction, in front of which nozzle, and within the hopper, there is a conical mouth in connection with an ascending pipe. The foul sand is thrown into two hoppers arranged at convenient points, one in each half of the bed, and the jets drive the sand, together with water, up the ascending pipes and into the first hopper of the circular series arranged in the centre of the bed, this hopper being provided with a screen to separate stones, etc. The material is thus carried forward, until finally it is delivered into barrows from the last ascending pipe in a clean state. Each hopper is provided with an overflow aperture from which a portion of the washing water overflows into the central basin, carrying with it the dirt, etc., removed from the sand, and a jet is provided at the bottom of the central basin so that any sand that may have been washed over may

again be passed through the washers to avoid waste of sand. The overflow from the central basin is controlled by a Weir penstock, and the waste tank is provided with sluice valve.

We now come to another form of water purification which is of more importance in water supplies for steam-raising and for manufacturing purposes than for domestic supplies—that of softening water. We have already made reference to the hardness and qualities of hard water, and pointed out how detrimental to some manufacturing and commercial purposes it is. We are now going to discuss how the evil can be rectified in a practical manner, in addition to our previous remarks. The process with which we are principally concerned is known as the Archbutt-Deeley process, and is world-renowned. For some of the drawings and for data supplied I am indebted to Messrs Mather & Platt of Salford, who so kindly assisted me. The experience of such specialists is always valuable. Let us first review a little closer the causes of the hardness, in addition to what has already been said.

Hard water, so-called on account of its soap-destroying property, owes its peculiarities to the presence of compounds of lime and magnesia, chiefly the carbonates and sulphates, but occasionally also the chlorides and nitrates. Hardness is generally distinguished as "temporary" and "permanent." "Temporary" hardness is caused by the presence in the water of the carbonates of lime and of magnesia, these compounds being dissolved by water containing carbonic acid in solution. If, therefore, the carbonic acid be removed by boiling or by precipitation by lime, the "temporary" hardness disappears—almost entirely in the case of carbonate of lime, and to some extent in the case of carbonate of magnesia; but to effect the complete removal of the latter salt, a sufficient excess of lime must be added to decompose the carbonate of magnesia into hydrate of magnesia, which is insoluble.

The presence of sulphate of lime in water, constituting "permanent" hardness, is due to its solubility even in pure water, and is not dependent on the presence of carbonic acid. Therefore the mere addition of lime will not effect its precipitation, but it must be decomposed by the action of some other compound, such as carbonate of soda. The hardness due to sulphate of magnesia can be removed by the addition of caustic soda, or, what comes to the same thing, and is cheaper, by the addition of lime and carbonate of soda in proper proportions; and as these two compounds are also wanted for treating the carbonate and sulphate of lime and the carbonate of magnesia, the actual chemical manipulation becomes very simple.

The mechanical operations involved in the softening of hard water include the preparation of the reagents, mixing them with the water, the separation of the precipitate produced, and the subsequent carbonating of the softened water. In this process it has been found possible by a special method of manipulation to increase the rate of subsidence of the precipitate to a great extent. This is effected by the simple expedient of stirring up, every time a tankful of water is softened, some of the precipitate which has settled from previous operations. This precipitate collects into coarse flakes at the bottom of the tank, and these, when stirred up and allowed to resettle, carry down with them the fine particles of the new precipitate.

Process of Softening.—The illustration in the folding plate (page 578) represents a two-tank plant suitable for treating quantities of 5000 gallons per hour, according to size. The plan comprises two cast-iron tanks, fitted up exactly in the same way, the processes of filling, softening, and clarifying

A gauge mark on the side of the softening tank, about 2 ft. from the bottom, indicates the level below which the water is not drawn off. Shortly before the water reaches this level the further descent of the discharge pipe is stopped by a "rest" fixed on the bottom of the tank.

The quantity of carbonic acid required is only just enough to convert the slight excess of alkali used in softening into bicarbonate, and it is controlled by a very simple test. No injurious effects need be apprehended from the presence of sulphur in the coke, for, assuming the coke to contain 2 per cent. of sulphur, even if all this sulphur were to be oxidised to sulphuric acid, the quantity of sulphuric acid per gallon of water would amount to only 0.055 grain.

Cost of Softening Water.—To remove carbonate of lime from water costs very little, because the addition of lime alone is necessary, and this is very cheap. To remove sulphate of lime, carbonate of soda must be used, which increases the cost. For the removal of magnesium salts, caustic alkali is required, and has to be used in greater relative proportion; waters containing magnesium salts to a considerable degree are therefore the most costly to treat, though, in the above method of softening, the caustic alkali being made as required by boiling lime and carbonate of soda together, is cheaper than if caustic soda were purchased ready prepared. The costliness of softening is in many cases a measure of the necessity for softening; and it is often more economical to soften a hard water, which can be bad for the pumping, than to purchase a town's water which is itself only less hard than the existing supply.

The varying character of water from different sources, and its effect upon the cost of softening, is illustrated by the table of analyses on p. 577. Most of these waters are under treatment at the present time.

Fig. 769A, the plate illustrating a water-softening plant, will explain the foregoing text. It is a two-tank plant, capable of dealing with 5000 gallons per hour, the soft water storing tanks being above the treating tanks.

being carried on in one, whilst softened and clarified water is being drawn off from the other.

For 2000 gallons per hour and less, one softening tank is sufficient if a storage tank be provided. For more than 6000 gallons per hour, three or more tanks are desirable.

The hard water is admitted to either tank by means of the supply pipe up to the level of a gauge mark on the side of the tank.

While the tank is filling, the quicklime and carbonate of soda are weighed out, and boiled up with water in the small chemical tank by means of live steam.

Some waters require also the addition of a little aluminoferric. This is conveniently prepared in the form of a standard solution, of which a measured volume is added to each tank of water immediately after the chemicals have been run in.

When the tank is full up to the mark, and the inlet valve has been closed, steam is admitted to the blower, causing a current of water to circulate up through the rose, the three way tap, down the vertical pipe, and back into the tank through the perforations in the upper row of horizontal pipes. The three-way tap is then opened and the prepared chemical solution is slowly drawn into the circulating current, and thus diffused throughout the body of water in the tank.

The air-tap is next opened to admit air through the pipe at the top of the blower, and the air is blown for a few minutes into the treating-tank through the upper pipes; then by reversing the three-way tap the air is forced through the perforations on the under side of the lower row of pipes. From these it rises in streams of bubbles, stirring up some of the precipitate or mud from previous operations which lies on the bottom of the tank.

After the blower has been in operation for a few minutes (usually ten, but varying with different waters) the steam is turned off, and in about one hour very nearly all the precipitate will have settled to the bottom of the tank, and the water, even down to a depth of 6 feet from the surface, will not contain on an average more than about one grain per gallon of suspended matter. The efficiency of the treatment can be readily controlled by a few simple tests, full instructions for making these being usually furnished with the plant by the makers.

Uncarbonated softened water is liable to form a deposit in pipes, and especially in the feed apparatus of steam boilers, which may become very troublesome. This is not a peculiarity of water softened by the above process, but is common to all methods of treatment.

The process of carbonating is extremely simple, and the extra labour and cost involved are inconsiderable.

The operations of carbonating and drawing off are automatically and simultaneously effected by means of the hinged pipe of rectangular section, the mouth of which is kept just below the surface of the water by means of floats. Fuel gas from the coke stove is forced continuously, by means of the small steam blower, up the pipe, escaping by the discharge mouth, and being caught by the descending current of water (which in flowing down the pipe is repeatedly splashed upwards by baffles fixed at intervals along the bottom) is carried along with the water through the ball tap fixed over the small supply tank, into which the softened and carbonated water falls, whilst the residual nitrogen, etc., of the fuel gas escapes into the air. If the small tank should fill up and cause the ball tap to close, the gas escapes through the vent pipe.

TABLE SHOWING THE VARYING CHARACTER OF WATER FROM DIFFERENT SOURCES, AND THE CONSEQUENT DIFFERENCE IN THE COST OF SOFTENING.

Source.	Well.	River.	River.	River.	Canal.	Well.	Brook.	Well.	Used Shaft.	Clay Pit.
Grains per gallon.										
Calcium Carbonate (Carbonate of Lime)	10	8.74	13.15	16.39	16.99	9.19	2.96	9.41	8.3	1.39
Magnesium Carbonate	4.76	2.73	3.83	3.1	2.76	1.4	3.4	1	2.82	1.78
Sodium Carbonate	4.19	3.26	1.96	4.3	2.69	12.17	47.34	22.91	40.61	51.14
Calcium Sulphate (Sulphate of Lime)	4.16	1.44	3	1.28	12.41	7.06	5.7	15.9	22.26	22.46
Magnesium Sulphate	...	...	...	...	15.26	...	9.98	...	2.65	28.98
Sodium Sulphate	...	...	...	small	...	13.69	...	11.5	small	...
Magnesium Nitrate	...	...	...	...	...	...	...	...	...	...
Sodium Nitrate	...	...	...	...	...	...	...	...	...	...
Magnesium Chloride	1.65	2.72	2.04	3.95	5.28	6.3	6.77	5.95	6.35	5.28
Silica	...	43	39	42	31	62	62	9	84	96
	24.75	19.37	19.15	25.75	54.7	51.06	73.41	68.75	83.86	114.37
Total Lime (CaO)	5.8	6.24	7.36	10.95	7.39	10.13	20.64	14.7	21.39	23.07
Total Magnesia (MgO)	2.23	1.33	1.81	5.8	5.15	7.92	2.36	9.82	8.81	8.38
CALCULATED HARDNESS (i.e., Total Lime and Magnesia, calculated to Carbonate of Lime)	15.65	14.5	15.16	20.99	26.77	35.53	42.7	50.57	60.92	61.95
Approximate Cost of Chemicals required for softening 1000 gallons.	0.2d.	0.5d.	0.4d.	0.6d.	1.2d.	2d.	2.8d.	3.1d.	3.6d.	4.2d.

NOTE.—The above estimates of cost are based upon the following prices, viz.:

Quicklime £1 per ton. 58% Alkali £4 per ton.

CHAPTER XXXIV.

WATERWORKS DISTRIBUTION.

We now come to the consideration of how the water in the storage and service reservoirs is conveyed to the consumer by means of pipes and conduits, and also how it is measured and controlled, and the general details of the scheme after final filtration and purification.

Conduits may be open watercourses, tunnels cut and cover (viz. the ground opened up, the conduit constructed and arched over, and the earth filled in), or raised aqueducts. We have already treated the flow of water in conduits, and have said something regarding their size and form; we will now add a few words on their construction. For a covered conduit fig. 770

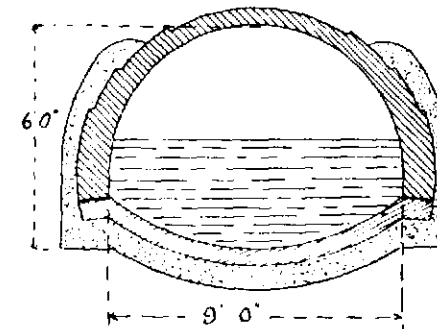


FIG. 770.

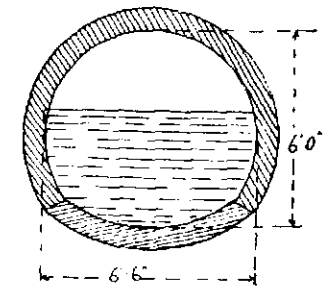


FIG. 771.

satisfies most requirements, being very strong, and affording a large hydraulic mean depth and an economical distribution of the materials. Open aqueducts do not demand much attention; their sides must be naturally firm to stand at the slope at which they are cut; and if this is not so, they must be pitched with stone, or otherwise lined and puddled with clay if necessary.

Another common form of closed conduit is shown in fig. 771. Aqueducts will frequently have to cross valleys, and when this is the case they give rise to the opportunity of much ingenuity on the part of the engineer. Some good examples are to be seen at the present day, notably the Barton Swing Bridge, which carries the Bridgewater Canal over the Manchester Ship Canal at Barton, about four miles from Manchester. They may be of masonry, cast-iron, or steel-plate construction.

By far the most common way, however, of bringing water to the consumer is by means of cast-iron pipes; and although they have their disadvantages,

they do not seem to have been yet superseded by any other method. It is true that for very large mains riveted steel pipes have given very good results, but the economy by their use is comparatively small. Unfortunately they cannot be allowed to be subject to any superincumbent load, and therefore are not of use for laying under roadways. In practice, cast-iron pipes may be successfully used up to 40 in. in diameter; they may follow the natural inclination of the ground surfaces and need no specially prepared bed. Trunk mains are sometimes laid in duplicate; above 40 in. in diameter there should be two pipes in any case, as larger diameters in cast-iron pipes are not advisable. The size of the pipe first demands attention, as stated under the heading of Hydraulics.

Having settled this, let us investigate the strength of cast-iron pipes. We know the bursting pressure in a pipe is proportional to the radius and the bursting pressure = actual pressure in lbs. per square in. $\times$ 2 radius in inches. Now in any pipe under pressure, the pressure $\times$ 2 radius should be equal to the allowable stress (maximum) $\times$ the thickness of the pipe (inches). But

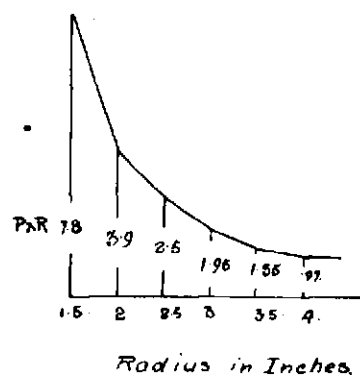


FIG. 772.

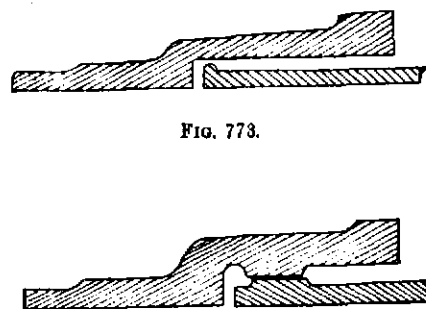


FIG. 773.

FIG. 774.

the pressure in a pipe acts only at a very thin film of the metal on the interior surface.

$\therefore$ the tensile testing of cast iron for such purposes is of very little value. The thickness of cast-iron pipes is obtained by a formula of which the most satisfactory is perhaps Box's, in which, if we are given the diameter in inches, and the head under which the pipe is to work under in feet = d and H respectively, we have the thickness in inches

$$= T = \left(\frac{\sqrt{dH}}{10} + 15 \right) + \frac{Hd}{25000} \quad (403)$$

the 25,000 being the cohesive strength of the material, but this is usually 15,000 lbs. per sq. in., but as H = ft. head it becomes 25,000.

A simpler equation is

$$t = .000192pd \quad (404)$$

t being in inches and d also inches, and p = pressure in lbs. per square in. Now the intensity of pressure in the metal varies as the square of the radius, and this is evinced by the curve fig. 772, in which the radius is plotted horizontally and the value $P \times R$ vertically. Cast-iron pipes are made

in lengths from 6 feet to 12 feet according to size. They must be cast vertically in green sand. The form of joint varies. The commonest form is the ordinary spigot-and-socket joint, shown in fig. 773. It is run up with lead, which must be afterwards caulked with special caulking tools.

To avoid, however, the unsatisfactory way in which such joints were often made, the pipes being frequently considerably out of line, the turned and bored joint, fig. 774, was introduced, in which there is a taper projection on the spigot end of the pipe, fitting into a corresponding bored recess in the socket of the next pipe. All that is necessary is a little red lead on the faces and the pipes driven home with a mallet. They are sometimes called "drive" pipes. The use of lead wool in spigot-and-socket pipes in place of molten lead has come into use lately, and is very satisfactory, the hammering of the lead making it practically a solid mass. It certainly saves time and labour. Large pipes and pipes for very heavy pressure would have flange joints. These joints may be faced and put together with a little red lead, or a rubber ring in a V groove may answer the purpose very well. The idea is shown by fig. 775. Mains for hydraulic power purposes working under considerable pressures would be, of course, very much stronger in construction, and instead of flanges two lugs would be cast on each pipe, with a rubber ring joint. These lugs would be set back an inch or so from the joint, a procedure which has

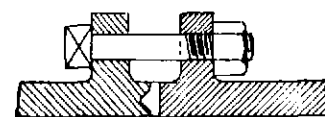


FIG. 775.

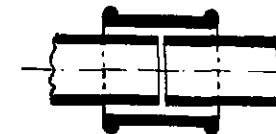


FIG. 776.

been found to add as much as 50 per cent. to the strength of the joint. Pipes which convey water for power purposes are often laid at a steep gradient down a hillside, in which case the riveted steel main is to be preferred. It must be securely fixed to anchoring blocks of concrete, to avoid "creeping" down the hillside. All water pipes must be coated with Angus Smith's patent solution; and also all valves, air-cocks, and any other iron fittings in connection with any water or sewerage scheme must always be so treated. The compound is a solution of pitch, linseed oil, asphalt, and resin, and is put on at a temperature of 300° Fahr. The manufacture of water pipes of any kind demands much attention, and the specification at the end of this book will give the reader much information on the subject which may be omitted here. The laying of a water main also demands a little care and attention. For heavy work, shear legs and tackle are necessary. They are usually laid on the natural bed of the trench, no concrete being usually required, but the bed should always be firm and excavated to the correct depth, and have as uniform a surface as possible.

The joints would be made as the work proceeds, and the matter of laying a main does not usually provide any real difficulties. At the end of a long line, or any line, it will probably be necessary to use a short length of pipe to finish up with; the joint is then made by means of a collar, which is shown by fig. 776, lead caulked, or the pipe may be sprung into position. Laying mains across rivers and ravines would be somewhat after the nature of laying sewer pipes, especially the means of slinging them on steel joists when possible.

Or they may be arched in form where crossing canals, provided ample head is present to overcome the resistance to flow. The thrust of the pipe must be duly provided for. Or take the following particular case of laying a water main across a tidal river (*vide* also Chapter XXXVIII.) :—

"In a dry summer the river is about 180 ft. wide, but about 200 ft. is the average width, with barely 2 ft. of water. The bottom of the river is subject to very little scour, the action of the tide compensating for flood scour. The river flows from south to north, and the tide runs from north to south. The gut, however, is liable to change its position. Some years ago it was close under the bank on the left-hand side, but some alterations to the river bank higher up caused the gut to move to its present position. The bed of the river from 200 ft. to 1000 ft. on section is of sand to a depth of about 20 ft.; but from 1000 ft. to the bank is now of a shilly nature, and is not subject to scour. The pipes must be laid about 4 ft. below the bed of the river, in order to meet the requirements of the Board of Trade.

"As the pipes have to be laid 4 ft. below the bottom of the river in the sand, there will be some difficulty in crossing the guts at each bank of the river; and as the work can only be carried on during low tide, the course of the gut will have to be diverted while this section is crossed. In order to divert the course of the gut an obstruction must be placed higher up the river. The right position for this obstruction will depend upon the course of the gut, whether winding or not. Should it be winding, there will be no great difficulty in temporarily changing its course, but the best position for the obstruction cannot be determined without inspecting the river. As practically the whole of the river bed is of sand and is liable to shift, the concrete slab containing the pipes should be carried on piles. The piles are cast in moulds in a horizontal position on the banks, as near to the site as possible in order to avoid cartage, and taken out to their position at high water in a barge, the piles being 9 in. square with four $\frac{3}{4}$ -in. steel rods. The rods are placed in the mould and adjusted by gauges to $1\frac{1}{2}$ in. from the sides; ties of steel wire $\frac{3}{8}$ in. in diameter are placed round the rods, spaced at the top and bottom at 6-in. centre, increasing to 12-in. at the centre of the length of the piles. In order to facilitate driving, the piles are erected in a vertical position when the barge grounds, and driven down to the right depth. If the current is not strong, the piles may be driven 30 ft. apart, and in pairs, with an outside width of 2 ft., thus leaving the slab to extend 6 in. over the piles on each side.

"The piles having been driven to the right depth, the concrete is stripped away from the upper end to the level of the under side of the pipe slab, so as to expose the reinforcing steel rods. The length of each section must not be longer than can be finished during a tide. The pipe would then be laid in concrete in the usual way."*

Testing water mains is usually done at the manufacturers' works, that is, testing by hydrostatic pressure, although large local bodies may possess their own apparatus. It essentially consists of a cast-iron table having two face plates. Between these two plates the pipe is fixed by screwing them up in a horizontal position, the joints being made with leather or rubber packing rings. Water is then pumped into the pipe through a hole in one of the plates, the pressure being registered on a pressure gauge. The several apparatus in connection with a water distribution scheme are as follows :—

1. *Stop Valves*.—These should be provided at the beginning of every pipe line, and are always of the gate or sluice type. They should also be provided

* *Building World*.

where branches leave the main, at the entry and exit to service reservoirs, etc. The usual form of valve is shown by fig. 777, and may have flange or spigot-and-faucet connections as desired, according to the type of main. In large mains of high pressure, the pressure on the gate will be considerable, and for this reason they are sometimes reduced in size below the full bore of the pipe.

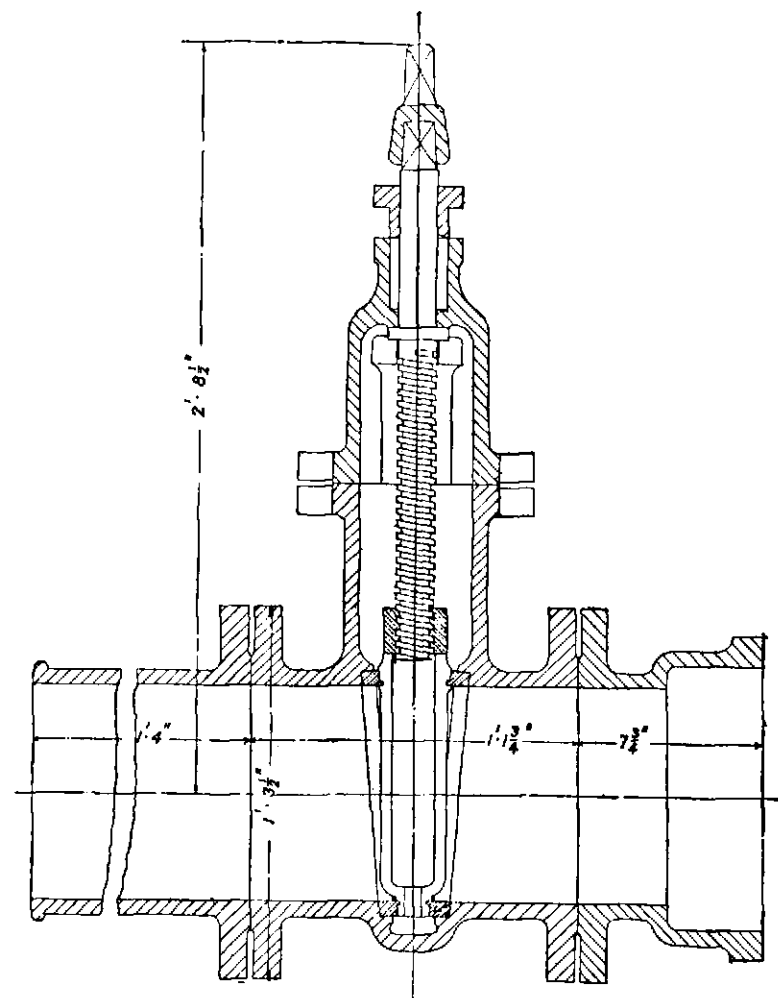
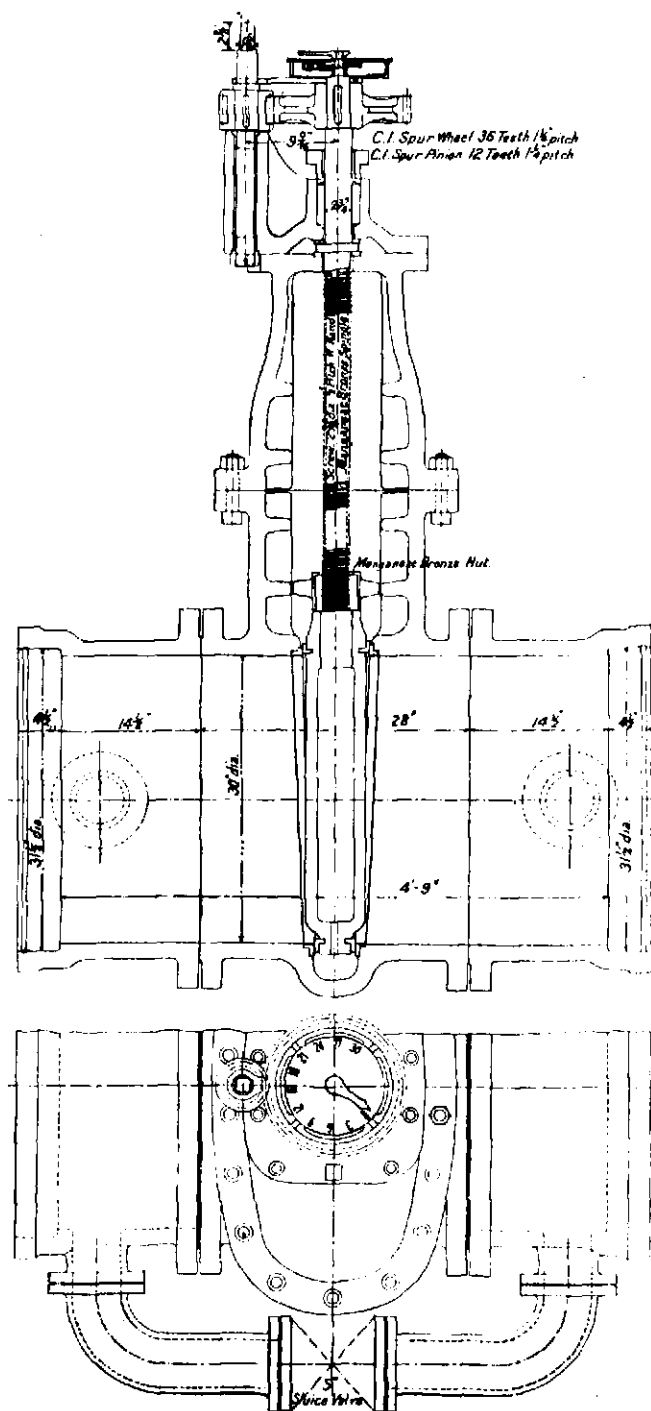


FIG. 777.

The practice, however, is not to be advised, and to avoid heavy pressures, causing difficulty of opening, we resort to three methods of overcoming the difficulty, which may be used separately or in conjunction with each other, and are as follows :—

(1) A by-pass valve of small area to equalise the pressures on each side of the gate.



FIGS. 778 and 779.

(2) Making the gate in two or more pieces, each actuated by a separate spindle.

(3) Worm, bevel, or other gearing to actuate the spindles. A good example of a type of valve working under these conditions is given in figs. 778 and 779, as made by Messrs Blakeborough & Co., Brighouse, who kindly supplied me with particulars. Spur gearing is provided, and a 5-in. by-pass valve.

2. *Air Valves*.—These valves are necessary at all points in a pipe line which form a summit, so Λ , because it would not be otherwise possible to fully charge the pipe after a shut-down, because of the air entrapped at the summit. A form of this valve is shown in fig. 780, and is usually placed in a manhole chamber near the surface on top of the main. It essentially consists of a gutta-percha or wooden ball, which, under ordinary conditions, rises on the water, and is held on its seat by the pressure of the same. When, however, the pipe empties, it will fall and allow of the air, which is driven forward by the ascending current of water, to escape till it will rise to the top of the pipe line. Slight leakage will usually take place when coming into action; this is particularly so when the balls have been in use for some time.

3. *Pressure Reducers*.—Where a town is served directly from a storage reservoir a great head may be encountered, especially in lower parts of the town and during night-time. If the mains and house fittings are not altogether of the best description, this is likely to cause much waste, which might not take place if the pressure was reasonably reduced. In any part of a district, 100 ft. pressure would be ample for most purposes where there were no high buildings. There are numerous different designs of pressure reducers on the market, but, on the score of simplicity, I have chosen for an example Broadhead's Patent, shown in figs. 781 to 784. I am indebted to the manufacturers (Messrs J. Blakeborough & Co., Brighouse) for the illustrations and description.

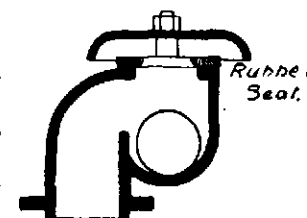


FIG. 780.

The inlet of the apparatus is at A; and when charging the main, the valve B opens by the pressure acting on the inlet side.

The piston C is loaded with weights to the required reduced pressure; and when the desired pressure on the outlet side is attained, the piston C moves upwards and lifts lever D, which opens regulating valve E, and closes regulating valve F.

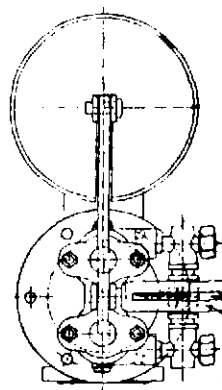
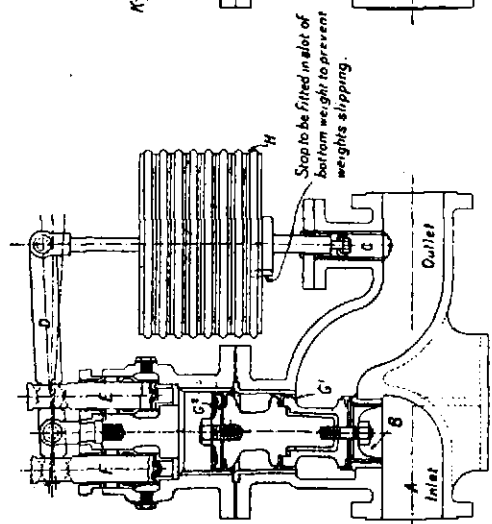
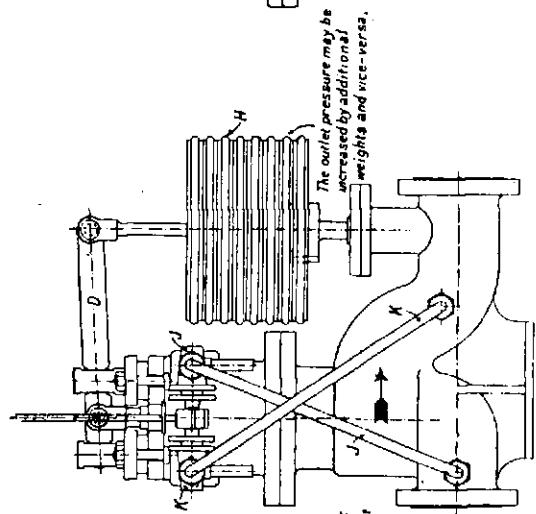
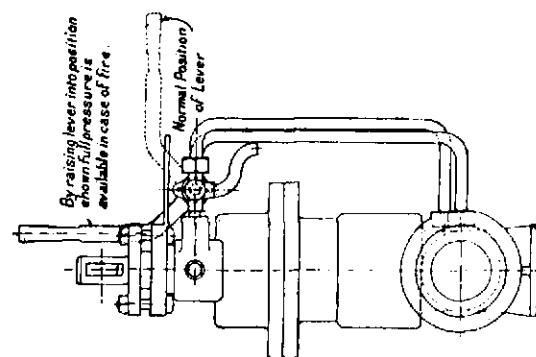
The regulating valve E being open, pressure on the inlet side acts on top of piston G², the water passing through auxiliary pipe and cock, or valve J.

The area of the piston G² being larger than the area of the valve B, the pressure acting on the piston G² closes the valve B.

The valve B remains closed until such time as the pressure on the outlet side is reduced by a draw on the main.

When the pressure on the outlet side falls below the pressure desired, for which the apparatus has been designed, the weights H overcome the pressure on the piston C, and the lever D falls and closes the regulating valve E, and opens regulating valve F.

The main being now charged, the valve B is acted upon at both sides, but by different pressures.



Figs. 781 to 784.

The pressure in the inlet side of the valve B combined with the reduced pressure acting on under side of piston G^1 opens the valve B and displaces the water at the back of piston G^2 , through valve F and cock or valve and pipes K, into outlet side of main.

For full initial pressure:—

In case of fire, the valves or cocks J and K are closed in the direction shown by arrows, etc., on the wheels on valves, or by an arrow on a quadrant for cocks, and $\frac{1}{4}$ turn of lever operating both cocks.

The valve or cock J cuts off the supply of water from inlet side to top of piston G^2 . The valve or cock K, being of a double acting pattern, cuts off the back pressure from acting on top of cylinder G^2 , and opens a port to atmosphere for emptying water from top of cylinder and allowing the valve to remain full bore for fire.

4. *Scour Cocks*.—At the bottom of a long depression in a pipe line it is advisable, if possible, to place scouring valves, because any deposit in the pipes which gets loose naturally will tend to deposit itself in the lowermost levels of the pipe line. Besides, at this point the pipe will more than likely pass over a watercourse, or there will perhaps be such a natural drain or a sewer near at hand into which the water from the main may be run. They are also of advantage in emptying the pipe in the case of repair. An ordinary branch pipe having a sluice valve will suit the purpose, but the mouth of the pipe should preferably be visible, so as the water men can see when clear water is delivered.

5. *Hydrants*.—These are special valves of large bore placed at suitable positions throughout a town water system. Primarily they are for use in case of fire, either to throw direct jets of water or to supply the fire-engines. But they also provide a ready means of filling water-carts, washing streets, etc., and other minor purposes. They are also useful for the same purpose as scouring cocks, when supplied with scouring keys, which allow the water to run out over the street, where it finds its way into gullies, etc. Each end of a pipe line which would otherwise be a dead end closed by a cap should have a hydrant on it, because much foul water gets trapped at this end, and if the main happens to be slightly large for its purpose, much of this water is liable to oscillate backwards and forwards in the pipe line and get into the consumer's premises, while by means of a hydrant the main can be periodically flushed. Hydrants are of various forms, the simplest obviously being advisable. They are usually made of the form shown by fig. 785. This consists of a cast-iron body or box containing a self-acting globular valve, which is lighter than water, consequently the pressure of water underneath it causes it to keep closed against its vulcanised india-rubber seating, so that the greater the pressure the more is the tightness of the valve secured. The top is made with two L lugs, so as to form a bayonet joint with the portable stand-pipe when it is attached, which is the best and quickest means of connecting the stand-pipe to the top of the fire-cock. Or, instead of the self-acting ball valve, a sluice valve may be provided, but they are naturally not so simple or rapid in action. All hydrants should have surface boxes, which the township men must always keep clean. They should be clearly designated by a

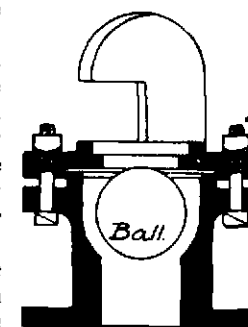


FIG. 785.

letter  on the walls of buildings, etc., preferably by white letters on a red ground, and referred to in a schedule of the water-mains map, giving their distance from certain known points. The valves should be treated in the same way.

6. *Reflex Valves.*—These valves are the same as illustrated in fig. 762, and are used on mains where a certain section is required to be kept charged and the adjacent section empty. With the self-acting type they will always retain the water in the main, which at times would perhaps be a disadvantage, to avoid which some are now made with a special screw-down valve, whereby they can be shut, or full or partially opened as desired.

7. *Relief Valves.*—To relieve excessive pressure and the effects of water hammer due to the sudden closing of valves or other causes. They are not much used on domestic supply mains, being mainly confined to hydraulic power mains, wherein sudden shocks would be set up due to the sudden stopping of the machines and the arresting of the accumulator ram. Relief accumulators are also employed (*vide Marks on Hydraulic Power*). The valves are on the same principle as the lever safety valve for steam boilers.

8. *Hatchways.*—These are rectangular oblong covers on pipe lines, usually 18 in. to 3 ft. long, and are provided to allow of scraping rods being admitted. They are screwed down by a number of bolts. The idea is shown in fig. 786, and should be placed in proper manhole covers. The scraping of mains, it



FIG. 786.

may be said, is often a very profitable procedure, when we consider the following recent examples:—

In 1907, 1400 yds. of 5-in. main at Berwick-on-Tweed were scraped, giving an increase of delivery of 50 per cent.

In 1900, at Pontypridd, about 4½ miles of 9-in. pipe and the same length of 10-in. pipe were so treated, with the result that, after the removal of sticks, stones, lead, cast iron, etc., a gain of 100 ft. pressure was obtained. The process, together with 3 miles of 12-in. main under similar conditions, cost about £275. We now proceed to consider house connections.

9. *House Connections.*—These are now practically without exception formed of lead or alloy pipes, of ½ in., ¾ in., and 1 in. in diameter, a large service being usually 2-in. to 3-in. cast-iron main. Gun-barrel is not to be advised for the purpose. Regarding lead pipes, the composition of the pipes and the water wants very careful consideration. Pure lead pipe, if suitable to some waters, will be so rapidly eaten away by others that lead poisoning will threaten the inhabitants. This is more particularly so with soft waters, lead pipes being absolutely inadmissible for conveying the Vartry water supply in Dublin and the surrounding districts, special alloys, having a large percentage of tin, being resorted to. These pipes are generally specified according to their weight per lineal yard, which will naturally increase according to the head they are to work under. For 300 ft. head, which it is usual to provide for, to be on the safe side a ½-in. pipe would weigh 7 lbs. per yd. and a ¾-in. pipe 11 lbs. per yd. To make the connection, the main is first

drilled. This may be done at night when the pressure is off, or by a special apparatus by which the work can be done when the pressure is on, of which some special patents are on the market; but the idea essentially consists of a cylinder fastened to the pipe at the point it is desired to make the connection, so as to make a water-tight joint. In this cylinder, and worked from the top, is a ratchet brace, which drills and also taps the hole, the cylinder in the meantime filling up with water. When the hole is finished the drill is screwed up and the whole affair slued round to bring a ferrule over the hole, which is screwed into the hole; the apparatus is then removed. The writer is of the opinion that better work is done when possible without its use. The ordinary ferrule to which the house service is attached is shown in fig. 787. The joint employed is the wiped joint. A special type for use with the pressure tapping apparatus is shown by fig. 788. The pipe is then continued up to the footpath opposite the premises it is desired to supply, where a stopcock is provided in a special surface box. The stopcocks should be made of gun-metal, and it pays to get the very best quality. A suitable form is shown in fig. 789.

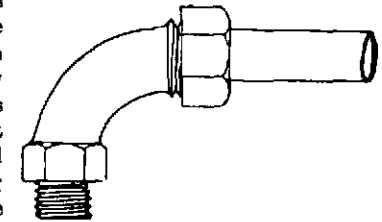


FIG. 787.

Metering Appliances.—We are now going to consider how water is measured

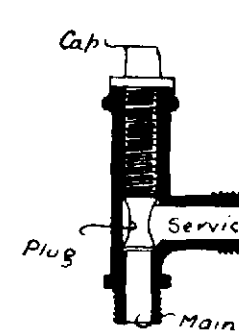


FIG. 788.

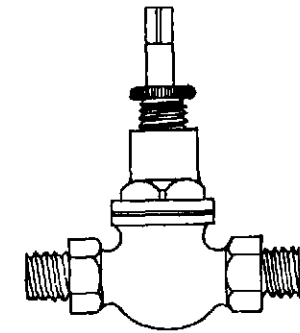


FIG. 789.

by mechanical contrivances in preference to weirs and notches, which have been already treated in Hydraulics, together with the various other ways of measuring water, confined to large volumes principally, and we now want to see how the small volumes of water can be measured by the gallon or smaller quantity. In some towns it is the practice only to have meters for recording the supply in large premises which use water for trade purposes, and to levy a water rate for ordinary domestic buildings; in other places every drop of the supply is paid for by meter, in the same way as electricity and gas. Each system naturally has its good and bad points. The use of meters in every place is naturally a fairly heavy expense, but at the same time the hire of the meter would probably be paid for, and the waste of water would naturally be cut down to the lowest possible limit in the consumer's premises. Again, the people may be so mean as to deprive themselves of the necessary quantity for general health. On the other hand, if

the water is only paid for by rate, people will be practically heedless of waste, necessitating a large staff of inspectors to see that such waste was not permitted, but it would ensure a plentiful supply being used by the people; and as the good health of the population is perhaps the main consideration, provided the water supply is ample to meet all requirements, things appear to be in favour of the rated supply, for dwelling houses at least. On the other hand, the use of water meters is by no means confined to buildings. Every district should have one, to enable waste to be detected; also the output of the reservoirs is usually so gauged, besides the delivery of the pumping engines, etc., and the work of the filter beds.

Of mechanical meters there are two main types, the inferential and the positive.

The former is on the principle of a turbine wheel, or rather Barker's mill, the wheel turning and recording its revolutions, a certain quantity of water being supposed to pass per revolution. The latter system is, however, quite different. Here we have one or two pistons, displaced by the water pressure, working in cylinders of a known capacity, and therefore delivering a known quantity at every stroke.

To be of any practical value a meter must be—

(1) Accurate at all flows; (2) cheap; (3) it must not cause appreciable loss of head in working; (4) it must be simple and easy of repair; (5) it must not wear or corrode rapidly; (6) occupy little space.

Of the meters of both classes there are numerous examples now on the market. It would be impossible to describe them all. Of the positive type, however, Kent's and the Kennedy meters are perhaps the best known examples; but to discuss makers' merits and demerits would not be prudent here, and for full descriptions of the several water meters now in use at the present day the reader is referred to the well got-up catalogues of the respective makers. Of the inferential type, the Siemens meter still holds the field; and as we are not concerned with the design or construction of such machines, it is somewhat out of place to fully illustrate and describe them here. Suffice to say, the engineer should select them under guarantee of reading correctly within 1 per cent., and, if he has time, should see them tested. Some authorities have specially calibrated tanks of their own for testing.

We only want to consider a few points regarding the art of metering. In the first place, there is no doubt that good metering stops waste, and it is exceedingly satisfactory for the engineer to be able to trace the flow of his water from the rainfall to the consumer. In fact, as a well-known authority on water meters puts it, they should measure from "drops to rivers." There is no doubt that metering an inadequate supply would probably do much to make it quite sufficient for many years to come.

Regarding the type of meter adopted, positive meters will measure the smallest flows, while inferential ones will not give so much accuracy. Again, the latter type are not so expensive in first cost, and are less likely to get out of order. From his own point of view, the writer prefers the Siemens type, because the working parts are very few. Others, however, would doubtless have reason not to care for this type, more or less a matter of choice and experience. At the same time the positive type *do* keep their accuracy, which cannot always be said about inferential meters. Again, we may have rotary piston meters. These are for somewhat larger flows than house supplies. They are accurate.

The reading of a meter correctly will often be a source of trouble to the beginner; in fact, it is surprising how many young engineers (and old ones too!) cannot read a meter. They certainly are a little perplexing at first. Take the following practical example, fig. 790 showing a typical dial. It is reading 988,899 gallons. Each small dial must be read by the light of that of the next lower value. In the illustration, that is to say, the 1000 dial hand is at 8, and not at 9, which might be supposed, because by looking at the large hand we see it is at 99. That is to say, when a pointer is between two figures, read the one of lower value. This will aid correct reading. A little practice is necessary at first.

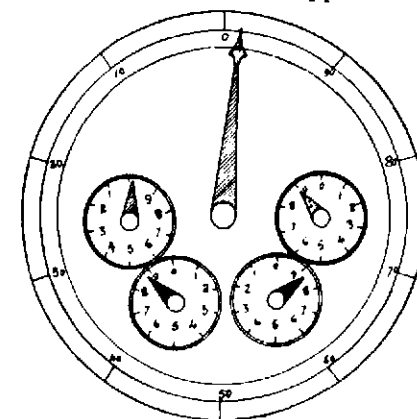


FIG. 790.—Reading: 988,899 gals.

For measuring large supplies of water, such as those passing through large mains, into and out of reservoirs and filter beds, the delivery of pumping engines, and the flow of sewage and other dirty and heavy liquids, an entirely different apparatus is used, and has given much success in the past, which is known as the Venturi meter. The sole European manufacturer is Mr George Kent, Holborn, London.

It is simple and accurate and durable, and we have already mentioned it.

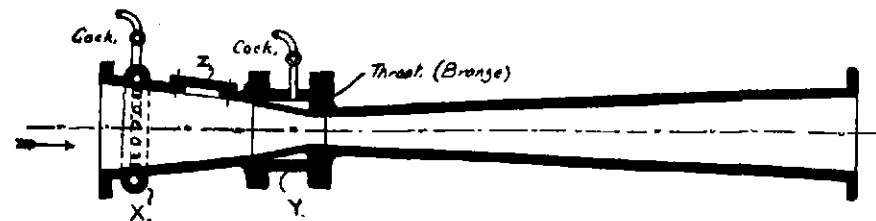


FIG. 791.

The Venturi law under which it acts is briefly that water flowing through a pipe of diminishing sectional area loses pressure, which it exerts laterally by gaining velocity, and the throat of the Venturi tube carries this so far as to be positively capable of causing a vacuum. The Venturi tube used in practice is shown in section by fig. 791. X is the annular pressure chamber (upstream side), Y = throat having a gun-metal lining with an annular chamber also, and Z a manhole for inspection. The accuracy of the gun-metal cone is most important; on it the whole apparatus depends. Now the Venturi head bears a constant relation to the discharge from the tube. Therefore, to get the quantity passed, we multiply the square root of this head by a certain coefficient. The register consists of a mercurial "U" tube. It is connected with the upstream side. The mercury in this tube actuates floats, which by means of rack-and-pinion gearing actuate a pen on a roll of paper, in the form of a daily or weekly diagram.

In connection with this meter, as with other diagram meters, there is an

apparatus called the *integrating drum*, so as to record the flow of the main

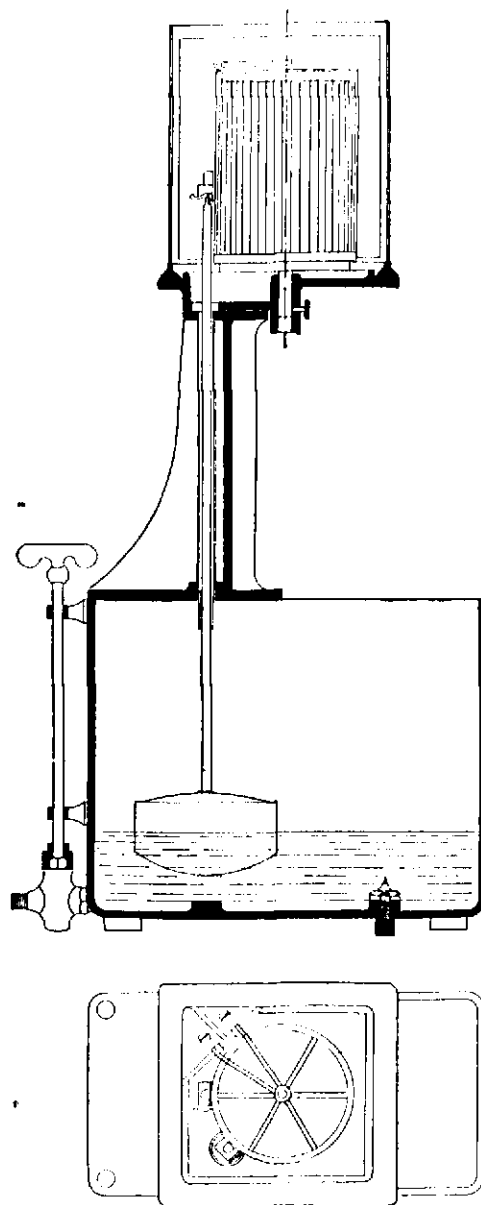


FIG. 792.

of observations have to be made, such an instrument is no doubt of the utmost value to the engineer.

numerically as well as graphically, and so save the tedious process, always liable to inaccuracy, of integrating the diagrams. A full description of the apparatus is given in Kent's catalogue. For low heads, such as measuring the flow in open channels, for which purpose the "Venturi" is particularly well adapted, water columns replace the mercury ones. The recorders of Venturi meters have the advantage of being capable of being placed at some distance from the actual tube, and, by means of electrical recorders, at great distances. For ascertaining the slip of large pumping engines there is nothing to beat an accurate Venturi, and every engineer of experience knows how important it is to reduce the slip to a minimum. Level recorders, applied to reservoirs, storage tanks, and the like, communicating over long distances with engineers' offices and pumping stations by electric current, cannot be overestimated. Over short distances, of course, any ingenious engineer can fit up mechanical recorders without much expense by means of a float, wire, pulleys, and a graduated scale. They need no description. A good form of recorder by the Palatine Engineering Co., Liverpool, is shown by fig. 792.

The instrument has been designed to record accurately liquid passing over a notch or weir gauge. The old method of taking observations by the hook gauge is then entirely obviated. When a long series

The electrical recorders are of a somewhat different nature. First of all we have the transmitter, consisting of an iron case with a glazed aperture; then, which is visible, a dial having two hands, which show by purely mechanical means the height of water in the reservoir over which it is placed, and forms a local indicator to which the distant indicator can be set. Inside, special apparatus make contacts (electrical) with every rise and fall, inch by inch, of the water, floats for the purpose being fixed in a special stand-pipe to prevent the effect of waves. At the office or other place we have the indicator, of which the internal mechanism consists of electro-magnets arranged to operate two pawls so as to move the hands backward or forward inch by inch according as currents are sent from the transmitter by the rising and falling of the

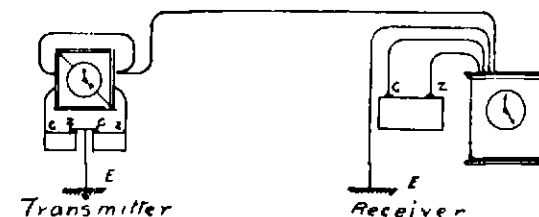


FIG. 793.

water in the distant reservoir. The shorter hand is arranged to denote feet and the longer hand inches by means of the graduations on the dial.

The whole mechanism is carried on a metal plate or framework, rendering all adjustments permanent. Inside the case are fitted two keys or buttons, marked "high" and "low" respectively; by pressing one or the other of

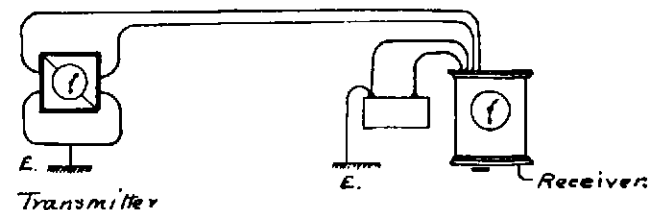


FIG. 794.

these, the hands of the indicator may be re-set at any time. This is rendered necessary by lightning strokes or other causes.

Two diagrams of the connections are shown by figs. 793 and 794, C and Z referring to the carbon and zinc poles of the batteries employed. Where the polarised relay is used, only one line is required.

The work should be preferably entrusted to an electrical engineer.

A relay may be defined as an electrically operated switch or push, according as its contacts are brought permanently or temporarily together. When a current flows round the coils of a relay and its armature attracted, two contact surfaces are brought together which close another circuit. The coils, armature back-stop and contact-stud are usually mounted on an iron frame. The armature, however, wants very delicate mounting (fig. 795).

The prevention of water waste is a matter of great importance to the municipal engineer; in fact, it is very often one of his most arduous tasks.

The ever-increasing demand for the precious liquid, and the corresponding increase (usually) of the difficulty and expense of obtaining it, have led to drastic methods being resorted to to cut down its misuse. In a certain town in Ireland, where the writer was borough engineer, all the water used is obtained and paid for from a separate water board, and at a high rate too (this applies to the particular district under consideration), under an agreement that when the statutory allowance has passed through the meters in any 24 hours, the officials of the Board in question have the right (and oftentimes use it, too) of shutting off the supply and leaving the district absolutely dry. Now, bearing in mind that the town has a large summer population, such an action in the height of summer (and that is the time when it usually occurs) results in nothing short of a water famine. Hence it was decided to install, at a fairly stiff outlay, waste-water meters of the differential type, which were introduced about 1880 by Mr G. F. Deacon, engineer to the Liverpool Cor-

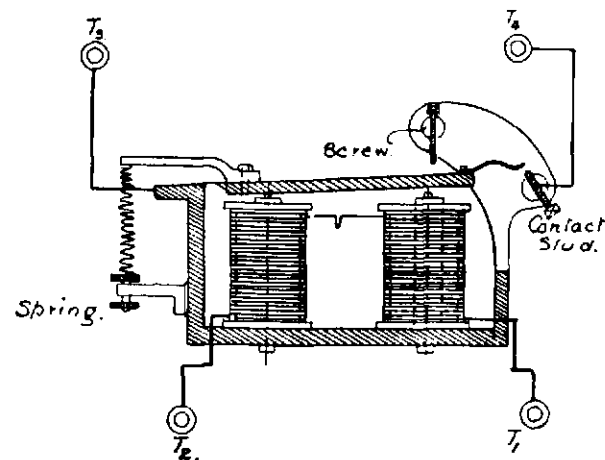


FIG. 795.

poration. The meters were placed in various positions in the district, together with the requisite valves, by-passes, etc. Referring to figs. 796 and 797, we have a section through the meter body. A is the cast-iron body, having two flanged ends, by means of which it is connected to the main. Fixed to this body in a vertical position is the hollow gun-metal cone B, so arranged that in its passage through the meter all water enters the cone from the bottom, and thence to the outlet, as indicated by the arrows. In this cone is a horizontal disc C, guided vertically by the stem D. This disc is of such size that when it is at the end of its stroke it actually fits the small end of the cone B. From the end of the stem D is a fine wire E, passing through a packing gland F to the dry chamber above, which contains the recording gear. This wire is connected to the carriage G, holding a pencil, the carriage being suspended over the pulley H by a flexible cord which connects to a counterbalance weight I. The weight is made a certain amount lighter than the disc, which therefore normally remains at the end of its stroke, with the disc exactly fitting the small end of the cone B. The movements of the pencil are recorded on a cylindrical drum K, round which is wrapped a metallic paper diagram,

divided by horizontal lines into gallons per hour, and vertical lines into time, which may be 6 hours, 24 hours, or 7 days, according to the clockwork actuating gear. When a flow takes place through the meter the disc is forced upwards and forms a waterway between the edge and disc of the cone, the

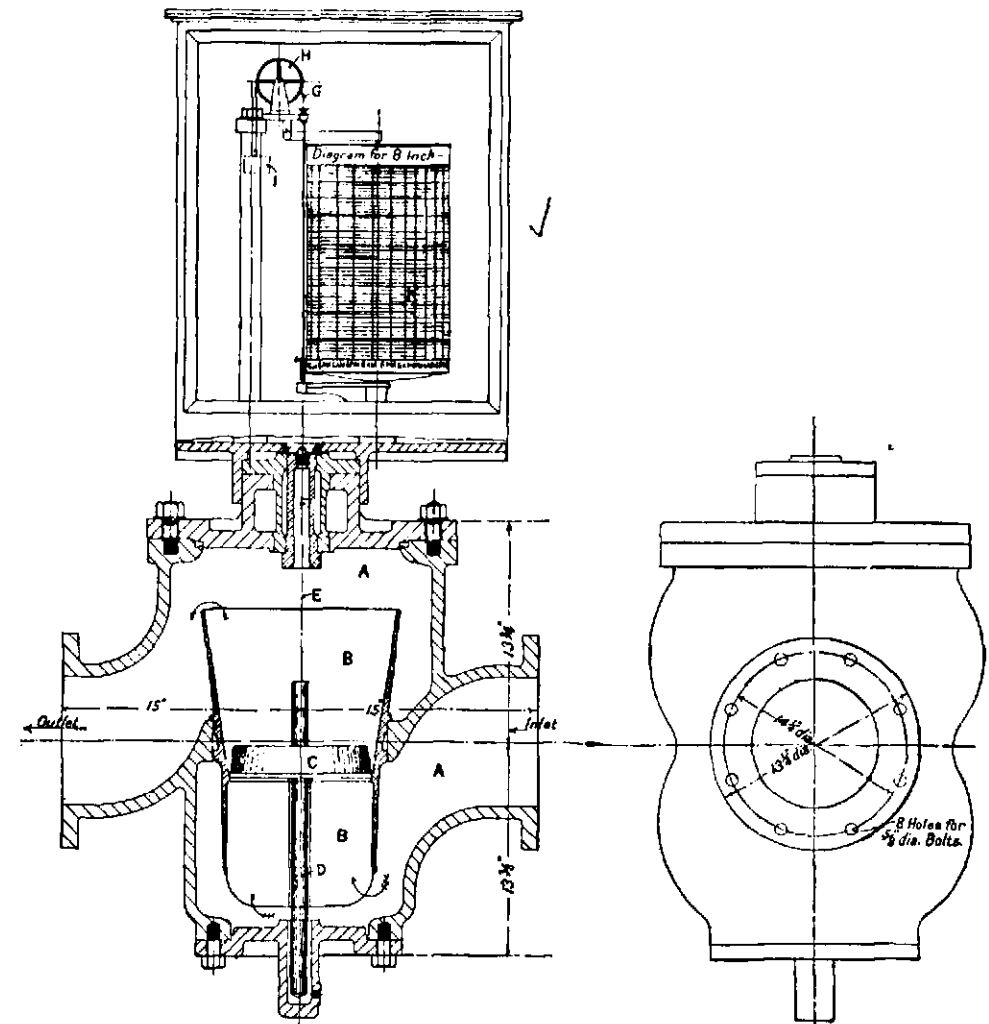


FIG. 796.

FIG. 797.

area of which is proportional to the amount of water passing. Suppose, for example, the rate of flow is 600 gallons per hour, the disc will ascend until the annular space between the disc and the cone will be just sufficient to pass water at this rate. It is clear that as the disc ascends the waterway the flow is increased, and *vice versa*; and it will be found that for any stated flow there is one position, and one only, in which the disc will rest, and consequently when

The advantage of the differentiating meter is that it will show the very smallest flows; and although there are now many well-known meters for actual measuring purposes, such as Siemens' and Kent's Venturi, etc., which have integrating and recording apparatus, for small flows the Deacon system stands supreme. The type of diagram in general use can be easily integrated to give the total consumption over a period of time by adding up the average values and multiplying by the time: 7-day diagrams are used for this purpose as well as the 24-hour; for practical use, however, the 24-hour diagram will give the most satisfactory results.

A very important problem which concerns civil engineers, especially those whose special duty is waterworks, is the rise of pressure in a line of pipes due to a valve being suddenly closed. The phenomenon is analogous to water hammer in steam pipes, which is the "bugbear" of pipe designers, and the cause of many accidents in steam-generating plants. Its cause is, however, somewhat different to that rise of pressure on water mains we are going to consider. It may be stated, however, that all whose duty consists in frequently closing and opening valves should be fully impressed not to do so rapidly. Let us take for example a riveted steel pipe working under a very high pressure for a pelton wheel installation of 500 ft. head, say the length was 1000 ft., and the valve was closed as rapidly as possible in about 10 seconds. Let us calculate the loss of head due to friction

$$= H_f = 4f \frac{l}{d} \times \frac{v^2}{2g} \quad (404a)$$

l and d = length and diameter of pipe in feet, v = velocity in feet per second, calculated on the head.

f = constant = .0076. If, however, we know that we have a flow of Q cubic feet per second,

$$v = \frac{Q}{\pi d^2 \frac{1}{4}} \quad (404b)$$

We can then call the loss of head

$$\frac{32flQ^2}{\pi d^5 g} \quad (404c)$$

by substituting. Now having found the actual loss of head, we must find the true head and calculate the pressure due to this head, call it P . This head may for the purpose of demonstration be assumed to be constant, therefore the velocity of flow is also assumed to be constant. Now in the length of pipe in question we have a certain mass of water having, by reason of its motion, a certain momentum, which is suddenly destroyed by the closing of the valve. Therefore mass of water in lbs.

$$= \frac{l\pi d^2}{4} \times 62.5$$

and momentum obviously

$$= \frac{l\pi d^2}{4} \times 62.5 \times v$$

$$= \frac{l\pi d^2}{4} \times 62.5 \times \frac{Q}{\pi d^2 \frac{1}{4}} = \frac{62.5}{32.2} \times \frac{Q}{\frac{1}{4}}$$

Now the valve cannot be closed absolutely instantaneously (it must take a small fraction of time), so say the time taken in closing = t seconds, that is, the time in which the momentum is destroyed. Therefore the "average force of the blow," as it is termed in mechanics, or better stated as "the time average of the force" (= the pressure of the water = P_1), which is required to stop the momentum in t seconds

$$= P_1 = \frac{62.5}{32.2} \times \frac{lQ}{t} = 1.94 \frac{lQ}{t} \quad (404d)$$

Then $P_1 - P$ = rise in pressure due to a rapid closing of the valve in question.

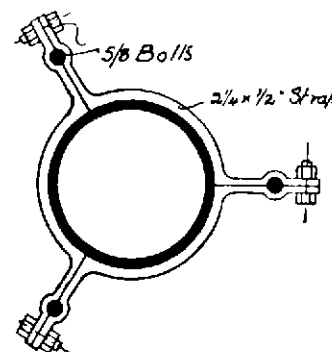


FIG. 798a.

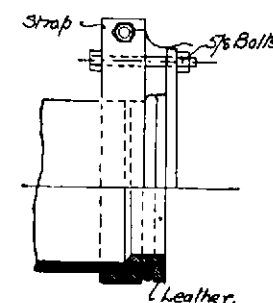


FIG. 798b.—Spigot End.

Although it is frequently specified that cast-iron water mains are to be tested as they are laid, the operation is fraught with so much trouble and difficulty that it is frequently allowed to be omitted upon the contractor giving an undertaking to locate and repair any leakage which may be shown on the completion of the work, by filling the mains and subjecting them to a superpressure at their highest point in excess of that which can possibly occur in practice. With reasonable care, unless the pressure is exceptionally high, there would be no leakage. If the ground is very undulating the mains would be laid in sections starting from the bottom of each valley on the pipe line and working up socket-end first to the top of each hill, where the pipes would be "married." If the gradients are not very steep it might be possible to start at the end furthest from the point of supply and lay the main straight forward, down hill and up, socket-end first, to the reservoir, but in this case lead wool should be used for jointing on the down grades, as molten lead would not fill up the back of the joint solidly. A contractor would probably want to start in three different places to expedite the work. Arrangements must be made according to circumstances for the supply of a large quantity of clean water for filling the pipes, and facilities provided for running off surplus water after testing.

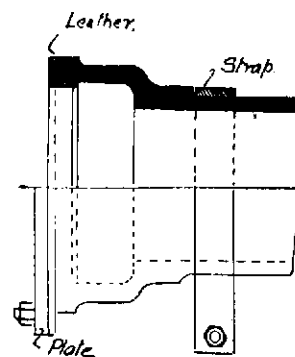


FIG. 798c.—Socket End.

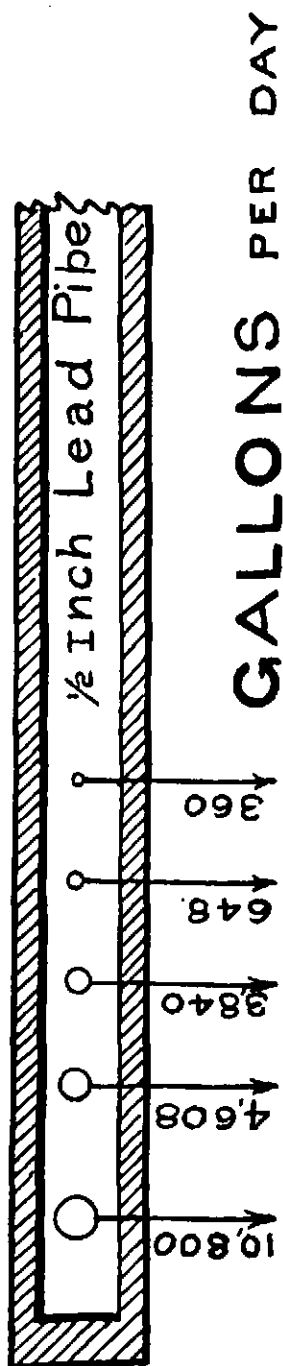


FIG. 798.

there is any increase or decrease of flow whatsoever it will be accompanied by an immediate change of position of the disc. As the pencil is connected to the disc, its every movement is reproduced and recorded on the diagram.

It may be well to state here that all such meters should be placed in a manhole where possible, and that manhole must be built water-tight, damp proof, and efficiently ventilated, and they are a great advantage especially when their depth will allow anyone to stand up in them. Frequently trouble will be experienced if damp is present, owing to the diagrams getting damp and torn by the pencil, besides the natural deterioration of the meter parts. The value of these meters, judiciously installed, cannot be overrated where used on a constant and high-pressure supply, because at night the pressure has a tendency to rise, and most of the wastage takes place then. Take, for instance, fig. 798, which is a full-size drawing of a half-inch service-pipe. Accurately drilled holes of the sizes shown were provided. Through each hole, the flow under a constant pressure of 45 lbs. per sq. in. was separately and carefully measured, and the illustration shows the number of gallons per day which passed. The test was carried out by the engineers of the Liverpool Corporation. The writer has records of a Deacon meter which registered a night flow of 5000 gallons per hour; and as there were at the time only four premises in the district in question on a metered supply, it was practically all waste. The rate paid for the water was 3.5d. per 1000 gallons. Taking six hours (12 midnight to 6 a.m.) we have 30,000 gallons, multiply by 3.5d. per 1000 = 105d.—nearly 9s. Every night during the previous years water was paid for at this rate, costing the ratepayers about $\frac{365 \times 9}{20} = £164$ per annum.

The population of the district was 7000; and assuming 20 gallons per head per day as an ample supply (the district was a poor one), we have 140,000 gallons. Again, if we assume the same night waste continued throughout the day, we have the stupendous figure of $5000 \times 24 = 120,000$ gallons per day running to waste, nearly as much as the actual legitimate consumption, due mainly to defective water fittings and incompetent inspection, and

in great measure, no doubt, to the inspectors being at a loss where to look for waste.

Now such a meter as we have described, if properly used, will show where the waste is principally going on, as will be presently pointed out.

Of course the figures given are somewhat excessive, because when full draw-off was going on it would be hardly fair to suppose that the actual 5000 gallons waste was continuing to flow the whole day through. The figures are only given to show what might possibly be the result of waste.

Now we can usually trace waste or nuisance to three distinct causes—

1. Leakages in the distributing mains,
2. Leakages in house connections and fittings,
3. Waste by consumers leaving taps running,

—section 2 being by far the largest.

Now the great advantage of the Deacon meter is that it enables a much smaller staff of inspectors to satisfactorily perform the duties, since it at once attracts special attention to, and indicates from, which particular cause (when intelligently used) the waste or misuse arises. In order to make a test of house fittings, an inspector must have a suitable form of stethoscope, which consists usually of a bamboo rod having an earpiece on top and a brass ferrule at the bottom. In the earpiece is a microphone arrangement, after the style of a telephone receiver. Of course, with any system for detecting waste or misuse of water, there must be means for subdividing the town into districts of moderate area for convenient inspection, and it is essential that each house should be provided with an outside stopcock placed below a cover inserted in the footpath, and accessible only to the officers of the town commissioners. Again, say we add the wastage of 120,000 gallons to the requisite daily supply of 140,000 gallons we have $\frac{260000}{7000}$ or nearly 39 gallons

per head per day. Now each of the mains on which there is a meter should have a sluice valve or valves below that meter, whereby any section may be shut down at will. Then the way to make a test by use of the instrument is as follows:—

In order to occasion the least inconvenience to consumers, the night should be chosen to carry out experiments, being also the time when proper use of the water is least. Each valve on the section should now be opened full for a short period and the water allowed to flow through it for a short time (the actual hour and minutes being carefully noted in the inspector's book in reference to the valve in question), then shut off, and so on with all the valves. When all valves are shut both at the beginning and end of the test the pencil on the diagram should run down to zero. A record of the quantity passing will then be obtained, and a diagram obtained by means of the 6-hour clock. Leaks in these particular sections will then be carefully localised by the inspector and made staunch, or notices served on users compelling (not urging only) them to have defects at once remedied. Particular districts should then be turned on, and the inspector proceed to make his examination along the line of the mains to which the house connections are attached, and shut off the stopcocks as he travels along the route, noting the exact time of doing so, and by means of the stethoscopic observations he notes whether the water is flowing into the several houses along the route. He also observes if, upon closing the stopcocks, these observations prove a flow of water at a particular house to continue. If this be so, it will indicate that a leakage exists between the stopcock and distributing main itself.

Again, the pressure will vary over every part of the system according to its level below the source of supply. Adjustable expanding plugs are quite unsuitable for stopping the end of the pipes; and if a cap were leaded on, it would have to be strutted tight to prevent its blowing off. After the test the lead joint should be cut out, which is a tedious job, and not burnt out, which destroys the coating of the pipe. The accompanying figures show a method of stopping the ends of the pipes. A small tap must be inserted in the stopper at the upper end to permit the air to escape when the pipes are filled with water. The pipes could be filled with water from a cart through a short length of hose, and the pressure produced with an ordinary small force-pump. Figures 798 A, B, and C show a good practical form of testing apparatus.

CHAPTER XXXV

BRICK AND STEEL CHIMNEYS.

WE now come to the consideration of large chimney-stacks. Such stacks are a common object to everyone, and are used in conjunction with steam boilers, principally in mills, power-houses, and factories in general, chemical works and refuse destructors. They are often of great height, and formerly were built of brickwork invariably, but now the practice of building steel chimneys and stacks of reinforced concrete has become very general, although in Britain engineers still adhere to brickwork firmly. Under the definition of "chimney," we can also take towers and such structures, when it becomes necessary to design them with a view of resisting wind pressure as the principal load upon them; in other words, when there is not any vertical load of any consequence which will fix the sectional area of the brickwork within such limits that the wind pressure thereon would have a distinctly minor consideration in regard to the internal stresses therein. About the middle of the nineteenth century the exigencies of mechanical engineers, who were making rapid strides with steam power, demanded that the height of such chimneys should assume large proportions, as evinced in the St Rollox chimney and Townsend's chimney in Glasgow—468 ft. high. The advent, however, of mechanical and forced draught for boilers tended to reduce this, and now the practical limit remains at about 200–300 ft.; but, as before stated, those in connection with refuse destructors are made very high on account of the necessity of removing bad gases away from possible harm. Very old chimneys on mills, when of only moderate height, will be found to be square on plan. The practice has now fallen into disuse. The reason is that the wind pressure sets up excessive stresses therein, and limits the height to under 100 ft. if economical construction is aimed at. The hexagonal form was then adopted, quickly followed by the octagonal, both of which have now practically given way to the circular, although the octagonal form still survives with some engineers.

Investigation of the effects of wind pressure on these four forms reveals the fact that if a circular chimney is taken as unity, the effective wind pressure on the surface of other forms will be

Octagonal	.	.	.	.	1.3	} . . . (405)
Hexagonal	.	.	.	.	1½	
Square	.	.	.	.	2	

Before considering construction, let us see what effect the masses of heated air passing through a chimney have in the form of what is known as draught. Say boilers of a certain horse-power (H.P.) are being installed, and that,

as a first approximation, we are assuming that the sectional area at the top = A sq. ft.

The required height of chimney in feet = H . Then

$$H = \frac{(7HP)^2}{A^2} \quad (406)$$

and conversely,
$$A = \frac{7HP}{\sqrt{H}} \quad (407)$$

Or say we know the height H , and the temperature of the flue gases = T , and the temperature of the atmosphere = t (Fahrenheit scale), and want to find V = velocity of the gases in feet per second,

$$V = 36.5 \sqrt{H(T-t)} \quad (408)$$

Obviously we shall have a certain draught therefrom in inches. Now it is this draught as measured in inches of water that is the important item to the steam user if he has no mechanical means of producing it. Therefore, say in a certain boiler installation a natural draught of d inches of water is required, the correct height of that chimney in feet

$$H = \frac{d}{\frac{7.6}{t} - \frac{7.9}{T}} \quad (409)$$

and conversely,
$$d = H \left(\frac{7.6}{t} - \frac{7.9}{T} \right) \quad (410)$$

Again, in place of using formula 408, let us find the amount of air passing through the chimney in lbs. per hour = P_A (.0059 lb. of air = 1 cub. ft.),* and let A_1 = mean effective area of chimney.

Then
$$P_A = d \times A \times \sqrt{H} \times 1000 \quad (411)$$

So much for these problems.

We must now turn to the more important question of actual design.

The wind pressure may be safely taken as horizontal, and the natural batter of the outside of the stack neglected with safety, to simplify the calculations.

There are various maximum values of wind pressure in Britain used by engineers. We have already suggested 40 lbs. per sq. ft. for use in roof calculations, but in such works as chimneys, in which the wind plays the most important part in the stresses thereon, a maximum pressure of 56 lbs. per sq. ft. would be advisable. Special values are taken in foreign countries, according to statistics or experiments.

Now in any chimney let a = the area of a vertical section across any diameter and p the greatest wind pressure.

The total pressure = P .

For square chimneys,
$$P = pa \quad (412)$$

For round chimneys,
$$P = \frac{pa}{2} \quad (413)$$

—in accordance with previous reasoning.

Then taking the moment of pressure about any horizontal joint at h feet

* At atmospheric pressure and 60° Fahr.

above that joint, the moment in lb.-ft. or tons-inches according to units taken = hP .

Square chimneys,
$$hP = hpa \quad (414)$$

Round chimneys,
$$hP = \frac{hpa}{2} \quad (415)$$

which must be checked over by the formula $\frac{W}{A} \pm \frac{M}{Z}$ to find out if the tension

and compression on the joints are in safe limits. Obviously the higher the column the greater will be the necessary strength of brickwork, due both to extra load and extra wind pressure. The foundations also will be materially increased. Failure to comply within safe limits with the condition $\frac{W}{A} \pm \frac{M}{Z}$ will cause two things—crushing of joints on the leeward side, or, more likely, opening of them on the windward. When a crack is so started it will usually extend, but its form is rather peculiar, inasmuch as it extends diagonally down each side of the chimney in zigzag form, following the lines of the joints of course. But this weakening of the joints on the windward side finally sends so much pressure on those on the leeward side that failure does *finally* take place, due to crushing of the brickwork on that side. Final collapse is, however, usually in the form of a complete breaking up. Let us take a chimney out of the perpendicular, as in fig. 799. Let d = its diameter as shown, and let x = the deviation of its centre from the original perpendicular centre line. Let q = ratio of x to d , while q' = ratio of the deviation of the point B from the middle of the diameter AC .

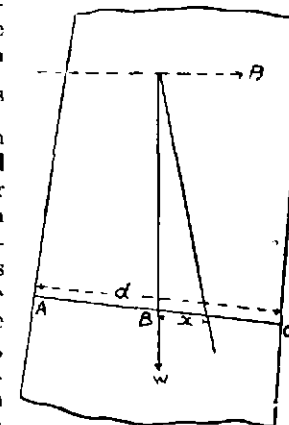


FIG. 799.

Then
$$Wx = (q - q')Wd \quad (416)$$

For practical purposes, however, $q = \frac{1}{2}$ for round chimney, and, for sake of comparison rather than design, $= \frac{1}{3}$ for square ones.

Reverting to equations 414 and 415, we have

$$hpa = \frac{1}{3} - q'(Wd) \quad (417)$$

and
$$\frac{hpa}{2} = \frac{1}{2} - q'(Wd) \quad (418)$$

It still, however, remains to find W .

Taking brickwork at 112 lbs. per ft.-cub. = W_a as before, B = mean thickness of the brickwork just above any joint under consideration.

Then for round chimneys,

$$W = 3.141Baw \quad (419)$$

and for square ones,

$$W = 4Baw \quad (420)$$

and combining with previous equations for square chimneys,

$$hp = \left(\frac{4}{3} - 4q'\right) Bwd \quad . \quad . \quad . \quad (421)$$

and for round ones, $hP = (1.51 - 6.28q') Bwd \quad . \quad . \quad . \quad (422)$

We can then decide upon the thickness of brickwork necessary for stability. As a guide, however, the following are the results of observations of actual practice. A chimney of 100 ft. in height, with an internal diameter of 5 ft. 9 in., would at the top have 9 in. of brickwork. By a uniform batter this would be increased to 14 in. at 20 ft. below the top, at 40 ft. to 18 in., to 23 in. at 60 ft. and 27 in. at 80 ft., or, in other words, the thickness

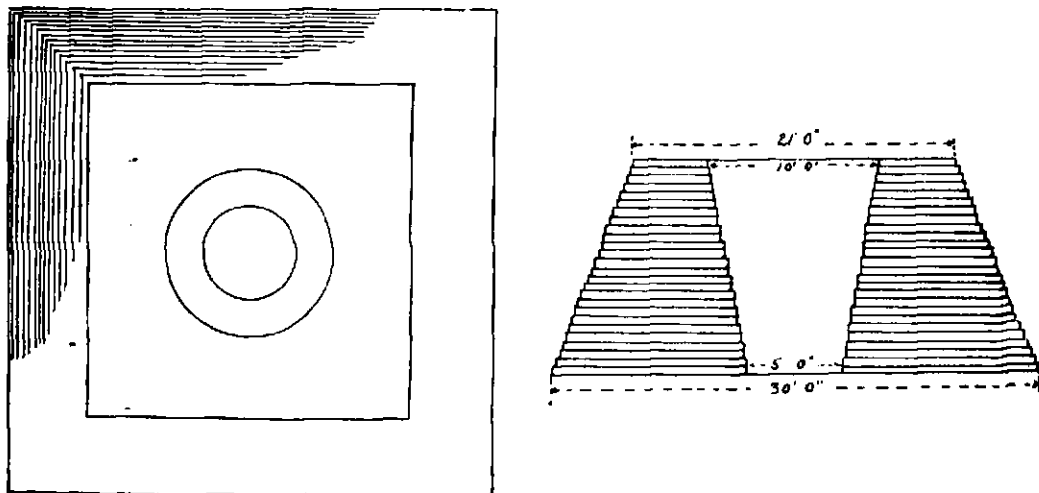


FIG. 800.

of brickwork is reduced $4\frac{1}{2}$ in., or a half brick, for every 20 ft. in height upwards. This applies to a circular shaft, but, provided that previous conditions are fulfilled, the rule may apply to other shapes. A mistake to be avoided in design is a heavy cornice projecting too far, which might be adopted for architectural reasons. No doubt such ornamentation looks nice, but, in these days of strenuous competition, is such expenditure necessary or advisable? Twelve inches is quite the maximum allowable in a medium-sized shaft.

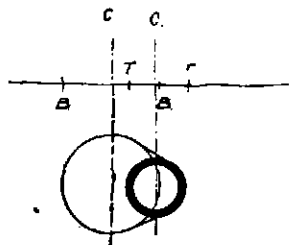


FIG. 801.

The foundation of a chimney is one of the important things. We have already (Chapter XXIV.) discussed the advisability of a dry foundation for this class of work. The footings for a chimney suitable for a stack of about 160 ft. high are shown by fig. 800. English bond is almost invariably used for the construction of the shafts, but specially-shaped bricks for use as headers are often used if very good work is aimed

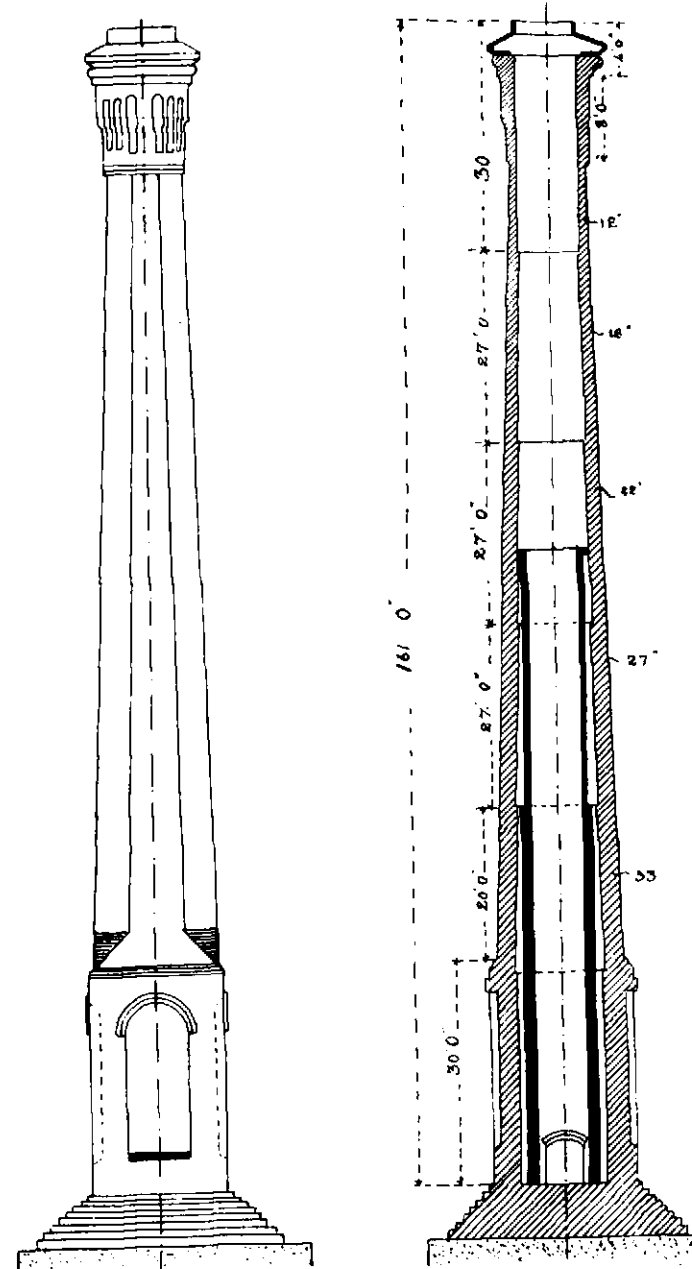


FIG. 802.

at and expense is no object. Such bricks certainly do make a better job and reduce the liability of bad joints.

The taking down and repair of large shafts is usually left to specialists in this class of work, commonly known as steeplejacks. Briefly the procedure consists of inserting wood (9 in. $\times$ 9 in.) studs at the base. They would be made about 5 ft. long, and when the chimney is safely bearing on all of them, a fire is prepared at the base of the stack so as to ignite the studs. If properly done by experienced men the stack should fall in a mass of brickwork at the base without danger to surrounding buildings. They usually, however, have to be stayed by guy ropes. It is a common occurrence for chimneys to get out of plumb. This can be rectified by experts. It is very often, however, the duty of the engineer to find the magnitude of such deviation. Using a transit theodolite, the method of procedure is as follows:—

Set up a theodolite so as to get a clear view of the maximum inclination. In this direction very carefully stretch a line on the ground, then take four observations, one on each side of the capping and one on each side of the base, and transfer the points on to the line very carefully, marking their position. The result will appear as in fig. 801, where TT represent the two marks transferred from the top edges and BB the two marks of the bottom edges likewise. By finding the centres of BB and TT = CC, CC will give the maximum deviation from the perpendicular. This process may be used to observe the sway of steel chimneys in high winds.

Fig. 802 shows a half section and elevation of a typical type of power-house chimney, with the dimensions marked thereon. The construction satisfies the conditions stated.

We will now proceed to investigate a graphical construction as applied to such cases. Take fig. 803, which shows a portion of a shaft. Let XY be any bed joint. Find by any usual method the centre of gravity K of the trapezoid AXYB. Draw a line PL to represent the wind pressure, the centre of pressure being taken as half the distance between AB and XY.* Draw LG to a convenient scale = total wind pressure and let GH vertical = weight of brickwork above XY. Complete the parallelogram GHML. GM must then pass through the joint XY so that NJ = x , when x is within safe limits (Equation 416).

We proceed to examine steel chimneys. This form of construction is now very prevalent amongst engineers, owing to the ease of erection of such structures. They have, however, only lately come into use in Britain, but American engineers have built them for some years. Speaking of steel chimneys, we are referring to those in which no guy ropes or stays are present, and on which the pressure of the wind is entirely taken by the steel plates composing them, and are commonly known as self-supporting chimneys, the strain due to the wind being entirely taken by the foundation bolts; and, provided the plates will successfully resist buckling, a marked saving of

* This is not absolutely accurate, due to the batter of the brickwork, but may be taken safely in practice.

material will result from proper design. We shall consider the chimney as a semi-beam uniformly loaded.

Let us take any section. Let D = diameter of the stack at that section, and d = diameter at a certain height above $D = H_1$. The overthrowing moment

$$= M_0 = 100 \cdot 5 H_1^2 (D + d) \quad (423)$$

in inch-lbs. We are assuming a wind pressure in this case of 50 lbs. per sq. ft. and a certain coefficient = .67.

This will give a very approximate result. If absolute accuracy is aimed at, it will be necessary to resort to the tedious process of finding the centre of gravity of each section and note its height above the base = H_2 . Then we construct a new formula for finding M_0

$$= M_0 = 201 H_1 H_2 (D + d) \quad (424)$$

inch-lbs., while the mean wind pressure on each section

$$= 33 \cdot 5 (D_1 h) \quad (425)$$

D_1 being the mean outside diameter of that section and h = height of section.



FIG. 804.

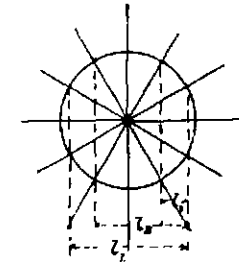


FIG. 805.

Then having a series of values of Equation 425, we can plot the results as in fig. 804. We refer the reader to graphic statics. In construction no vertical joints should be over each other; the seams, however, are only subject to crippling stresses. Now on the top section of the chimney we have no strain, but the least practical size of plate we would use for the purpose would be $\frac{1}{4}$ -in.; rivets would be $\frac{1}{2}$ -in. diameter at 2-in. pitch. We shall also probably be able to keep the same size of plate right down the shaft, but in order to do so we shall have to greatly increase the number of rivets in each joint, and that is the reason for the bell-mouth shape always adopted at the base, because even by double riveting it would be difficult to get the required number of rivets. The design is proceeded with as follows:—

1. Determine the size and height of stack.
2. Draw the centre line and mark off each plate section. For economy's sake they should be made all the same height, and large plates used to avoid too many vertical seams.
3. Mark the centre lines of all horizontal seams.
4. Through the proper points draw the outlines of the shell, making each

succeeding diameter larger in the correct proportion. The increase should be uniform. We are at present neglecting the weight of the stack.

5. Consider the foundation bolts.

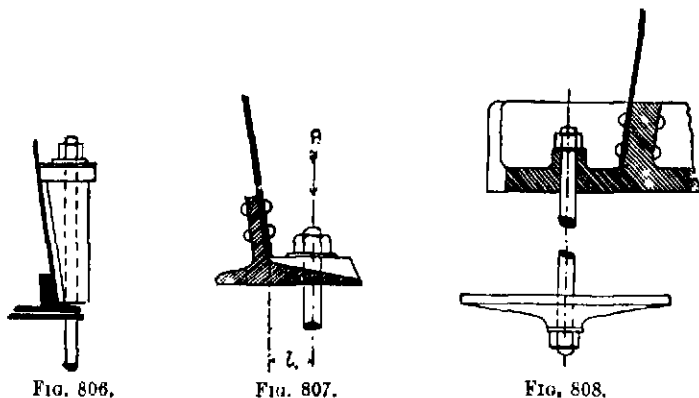
Let l_1 , l_2 , and l_3 be three values of the vertical lap in inches, as shown by fig. 805.

Let $M_r = M_0$ = moment of resistance.

Let P = load on bolt section.

Then
$$= \frac{M_r}{6(l_1 + l_2 + l_3)} \quad (426)$$

from which we proceed to find the correct diameter of the bolts, and the number of bolts will be settled by the designer to avoid large dimensions. We may then design the footings as shown in figs 806, 807, and 808. The latter two forms are usually made of cast-steel sections, there being as many sections as bolts. They require, however, careful design in accordance



with engineering principles. We must also find the required thickness of metal in that part of the footing to which the shell is riveted. In it we have shear and tension set up by the rivets, and also a bending moment from the foundation bolts to consider.

Now find the net area of rivet-holes in square inches = R_a ,

$$= t(\pi D - h d_2) \quad (427)$$

D being the outside diameter of the seam in inches, and d_2 = diameter of rivet holes in inches. Divide this into half the load, and we have the tension set up in the cast-steel ring which must be within safe limits.

Again, let a = single plate area of one seam. Then

$$a = \pi D t \quad (428)$$

D being the outside diameter.

Let S = unit stress on plate in lbs. per square inch for tension and compression; then the total strain on one seam in lbs. = T and

$$T = S a \quad (429)$$

and if there were two rows of rivets, half would come on each row. Equation (427) is apt, however, to give rather low values when the bending moment is

excessive, in which case it would be advisable to take the modulus of section into account. Now in a steel chimney of the type we are describing the resistance to collapse is provided mainly by the weight of the base, because we are not considering the weight of the metal nor the firebrick lining, so as to be well on the safe side.

If c = constant = .67

H = height of chimney
 D = diameter
 B = breadth of base
 W = required weight of chimney to ensure stability,

$$W = c \frac{DH^2}{B} \quad (430)$$

But the least practical value of W should be $3W$, which has to be provided by the base. Taking brickwork at 112 lbs. per ft.-cube, the effect of this has then to be transmitted through the foundation bolts.

Bolts have been already discussed in Equation 426. But for simplicity's sake, the formula 431, using the same units as the previous one, may be adopted,

$$P = \frac{C \times D \times H^2}{2} \times \frac{W \times B_1}{2} \quad (431)$$

B_1 being the diameter of the base plate, whose weight is not included in W . P should not exceed 8000 lbs. per sq. in. in this case. The steel base plate sections need not be bolted at the joints. The anchor plates are important. They should not bear more than 10 tons per sq. ft. of area, and to avoid overcrowding of these plates the bolts often have to be made in two lengths. In fig. 807 will be seen two values, P and l . Knowing them, the thickness of metal at the bolt-head is found by

$$T = \frac{Pl}{2} \quad (431a)$$

This applies to the foundation plates.

Fig. 809 shows a design for a stack on the foregoing principles. In steel chimneys there is a practical limit to size and height on economical grounds, and in large power-stations it is usual to see very often two similar stacks side by side. There is usually a light ladder running the whole height. The fire-brick lining is usually carried further than shown in fig. 809.

We have dealt with the draught in a boiler furnace, and it is very often the duty of an engineer to see what the actual draught is.

The commonest method of measuring the draught in a flue is by a water gauge. This consists of a U-tube 3 or 4 in. long, made of, say, $\frac{1}{4}$ -in. glass tube and graduated in inches. Test holes should be made through the brickwork at the boiler side of the side flue damper, economiser inlet, and chimney base. A reading should also be taken over the boiler fires. To use this gauge, a piece of $\frac{3}{8}$ -in. gaspipe about 6 ft. long is required, and india-rubber tubing to connect this to the water gauge. When the pipe is inserted in the flue, there will be a difference noted in the levels in the limbs of the water gauge. This level represents in inches of water the pressure in the flue below atmosphere. The pressure thus obtained will not give the quantity of

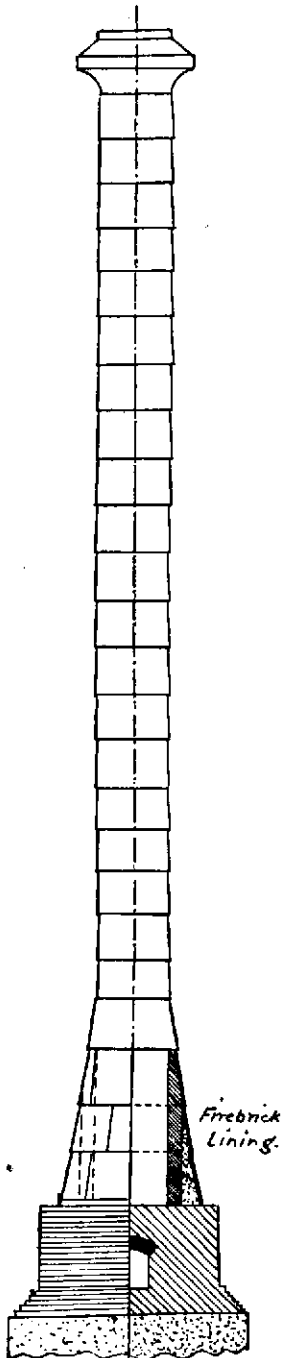


Fig. 809.

gases or the velocity of them, although it is approximately proportional to the velocity. A wheel anemometer or Pitot tube anemometer would be necessary for getting the exact quantity of air used, or flue gases. The draught varies considerably in different boiler plants, and will be as low as 0.20 in. in the side flue at some places, and as high as 1.00 in. in the side flue at others. In a well-designed boiler plant with, say, 0.80 in. chimney draught, the following draughts would be obtained: chimney base, 0.80 in.; economiser inlet, 0.65 in.; boiler side flues, 0.55 in.; over fires, 0.30 in. This would be the case with, say, three boilers fixed and working. The lowest draught at which it is possible to keep up the steam pressure is the best, the draught being regulated at the chimney damper. By doing this the speed of the gases is reduced, and consequently more heat is taken out of them before going to waste up the chimney. Bad draughts are generally caused by insufficient area of flues. The worst place, of course, will be found by the water-gauge readings at the different points, and will be where the greatest difference occurs in the results. Narrow economisers obstruct the draught, as do sharp corners, right angle bends in the flues, etc. Air leakages through the brickwork, round the boiler shells, and round the flue doors, etc., also have a bad effect on both the draught and the fuel economy. A steam jet under the bars reduces the quantity of air going into the fires, but assists the fire to burn bright and keeps the bars cool. The height of the chimney is important, but the best height can only be found by experiment.

Any accumulation of flue dust of course reduces the flue areas, and this should be removed often. If there is more than one opening to the chimney, a vertical wall should be built between, to prevent baffling effects between the two currents. In conclusion, it is always advisable to keep a thick, even fire, both for economy in fuel and improvement in draught. The apparatus is shown in fig. 809A.

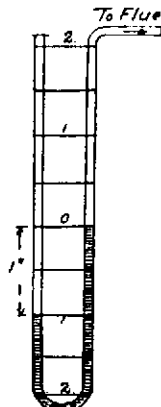


Fig. 809A.

CHAPTER XXXVI.

STEEL AND IRON CONSTRUCTION AND DESIGN.

WE now come to deal with a very important and comprehensive branch of the engineer's profession, that of designing structures in iron and steel, and the erection of the same in accordance with scientific principles. We say steel and iron, but the use of wrought iron in constructional work has practically fallen into disuse, steel being universally used, principally because of the economy due to its superior strength. The use of cast iron is still in a small measure retained for pillars of small dimensions, but its use in beams may be stated to have ceased. We would advise the reader carefully to lay to heart what has been said in Chapter XIII. on the properties of iron and steel, and also the principles enumerated in Chapters VII., VIII., IX., and X.

The object of the present chapter is to point out to the reader how these principles are put into actual practice. We are generally concerned also with the heavier branches of constructional steel, leaving light work such as hayricks, fencing, barns, and sheds to the architect; but we do not confine ourselves entirely to very heavy work, but rather aim at treating the subject so far as it concerns the municipal engineer. To start with, we proceed to investigate the scientific principles underlying the use of rivets and riveted work in general; we shall then discuss the properties of bolts.

In up-to-date shops, nowadays, the members of structures which require holes in them for riveting are drilled in the same way as parts of machines, the process being greatly facilitated by the special high-speed tool-steels now on the market. The practice of punching the holes is now considered very second-rate workmanship, and should not be allowed on any high-class contracts.

All holes should be drilled to templet. Then, when drilled, the two plates or bars to be joined should be bolted together and the rivet-holes reamed out $\frac{1}{8}$ in. larger than the actual rivet. No drifting should be allowed under any circumstances. The plates are then taken apart and any burrs cut off with a chisel. The holes must then have their outer edges gone over with a broad countersink. Riveting is done as far as possible in the shops, and should be done by a hydraulic riveter, which should always be provided with a special means of hydraulically closing the plates. That riveting work which is done on the job usually has to be done by hand, but on large bridge contracts the installation of a self-contained hydraulic riveter, electrically driven, may be deemed justifiable. The rivets so driven are called *Field rivets*, and must be specially considered in ways we shall describe. Such work wants rigid inspection. It is most essential that skilled labour be employed throughout the work.

Regarding the rivets used, the material of which they are made is an important consideration. When steel rivets are used they must be of the best quality—open-hearth steel of an extra soft quality; but the ultimate strength must be greater than 45,000, and not more than 55,000 lbs. to the sq. in. in tension, while the elastic limit should be at least equal to half the ultimate strength, while the elongation should be equal to $\frac{1,400,000}{\text{ult. strength}}$.

The maximum percentage of phosphorus allowed is .06 per cent. and sulphur .04 per cent. All bars from which rivets are to be afterwards made should be tested as follows: A piece of bar heated to dull red and plunged into water should bend 180° flat on itself without sign of fracture on the outside of the bent portion. When a rivet is purposely fractured it should show a close-grained silky fracture; coarse-grained rivets, showing a dark dull lustre, must be rejected. The specification for rivets should state that the bars when tested should have an elongation of at least 25 per cent. in 8 in. For field rivets, however, many engineers prefer the use of wrought iron. When used, however, it should only be the best quality Lowmoor (Yorkshire) iron, no other brand being admissible. Rivets should be heated in a coke fire; and when used for hydraulic riveting, be heated to a cherry red, but for field rivets they must be yellow. This applies to all hand work. Again, the pressure applied by the hydraulic process must not be either excessive or too little. About 100 tons per sq. in. of rivet area is enough. On long pieces of riveting it is essential that those in charge see that when the plates are together the two end rivets and then the centre one

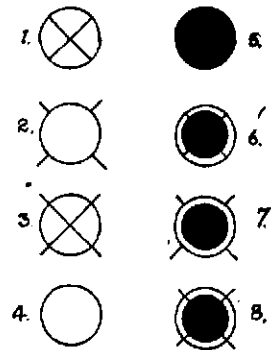


Fig. 810.

are put in first, closing up towards the centre afterwards. Riveter dies are usually made of high-speed air-hardening steel. For holding up, the compressed-air holder is the best. A very common fault found in riveted work is that of a rivet-head not being central. It must be carefully guarded against. It is principally caused by the rivet being too small for the hole. Hence the necessity for standard sizes of all tools and appliances, and the disfavour of punching; all such work must be rejected. Now in works and drawing-offices, where much of this class of work is executed, the different kinds of rivets and their actual position in the work as denoted by the drawing is very often somewhat confusing, although an old engineer can generally decipher them by reason of his experience. In order, however, to make matters clear and beyond subsequent dispute, engineers have adopted conventional signs for the various classes of rivets, which we illustrate in fig. 810.

- No. 1. Snap head on far side of plate.
- No. 2. Countersunk head on far side of plate.
- No. 3. Countersunk head on near side of plate.
- No. 4. Snap head on both sides.
- No. 5. Parallel hole. No rivet therein.
- No. 6. Countersunk on far side. (Hole only.)
- No. 7. Countersunk on near side. (Hole only.)
- No. 8. Countersunk on both sides. (Hole only.)

Now, in designing any riveted work, do not make the total thickness of

plates riveted more than five times the diameter of the rivet. The young designer must also be very careful to consider the actual work which the drawing represents, which condition, like so many others, make a shop training such a desirable, if not essential, qualification. For instance, a very common error is not allowing sufficient space in angle and tee irons for the riveter dies, and designing work so that the riveter has to work left-handed and in awkward positions. It is these little considerations which make such a vast amount of difference in the value of the engineer (practical) and the mere theorist. All rivets which are countersunk when cold should be chipped level. It will reveal bad work or slackness. It will not, however, as is sometimes supposed, cause the slackness.

We proceed to illustrate to the reader some eight varieties of riveted work. Fig. 811 shows an ordinary piece of riveted work.

Here we have the diameter = half the pitch, thereby reducing the plate area 50 per cent.; in other words, what is termed the efficiency of the joint is equal to

$$\frac{P-d}{d} \quad \dots \quad (432)$$

This efficiency is always allowed for in riveted work.

Rivets and Rivet Holes.—Usually the rivet is $\frac{1}{16}$ in. less in diameter than the hole for $\frac{1}{4}$ -in. rivet and upwards. All holes for rivets should be drilled after the plates have been bent or flanged and put together in their proper places. The diameter of rivet depends upon the thickness of the plates, and Unwin has given the following formula, in which t = thickness of the plate and d = diameter of rivet for boiler work:—

$$d = 1.2 \sqrt{t} \quad \dots \quad (433)$$

From $1\frac{1}{4}$ to $1\frac{1}{2}$ times the diameter of the rivet is the length that must be allowed for forming the head. The thickness of the head may be from 0.6 to 0.7 times d , and its diameter about 1.6. In boiler work the head is usually rather larger than in girder work.

Maximum Pitches for Riveted Joints allowed by the Board of Trade Rules is given by the formula

$$p = (k \times t) + 1\frac{1}{2}; \quad \dots \quad (434)$$

Number of Rivets in one Pitch.	1.	2.	3.	4.	5.
For lap joints k	1.31	2.62	3.47	4.14	...
For double butt strap joints } $k =$	1.75	3.50	4.63	5.52	6.00

p = maximum pitch.

Diagonal Pitch of Double Riveting.—Allow not less than 30 per cent more sectional area between rivets taken in a zigzag direction than in straight line:—

$$p_1 = 0.65 p + 0.35 d; \quad \dots \quad (435)$$

The *Board of Trade* rule gives slightly less than this—

$$p_1 = 0.6 p + 0.4 d; \quad (436)$$

p = pitch of rivets, p_1 = diagonal pitch.

Now a rivet joint may fail in a variety of ways if these rules are not adhered to. Figs. 811, 812, and 813 show two common ways; fig. 814 shows failure by crushing, and fig. 815 by shearing out the lap.

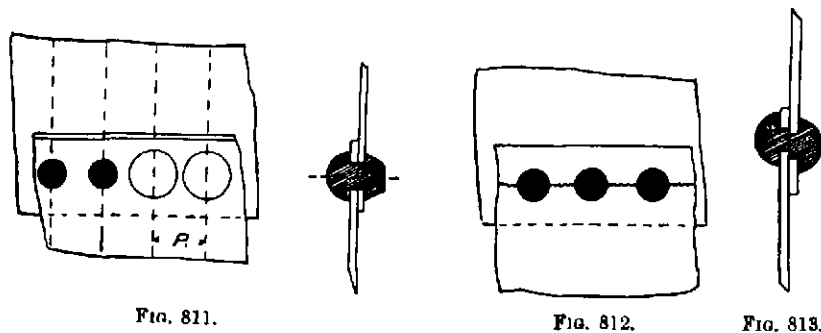


FIG. 811.

FIG. 812.

FIG. 813.

All rivets should be so driven that the distance from the edge of plate to edge of rivet is $1\frac{1}{2}$ times the diameter of that rivet at least.

Another formula for efficiency of a rivet is, if—

A = sec. area of rivet = $(d^2 \times .7854)$.

S = safe shear on same = 20 tons per sq. in.

t = thickness of plate.

f = tensile strength of same.

p = pitch.

$$\text{Efficiency} = \frac{As}{Pft} \quad (437)$$

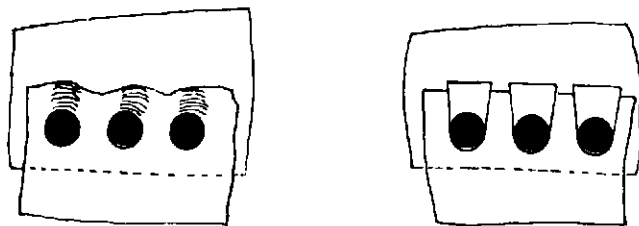


FIG. 814.

FIG. 815.

In addition to the single-riveted joint, we can, of course, have double-riveted joints (fig. 816) and treble-riveted (fig. 817), in which cases the pitch may be reduced as before tabulated.

We have been considering lap joints.

The butt joint will make a more satisfactory piece of work if properly done.

Fig. 818 shows one double-riveted and 819 treble-riveted. Now such

joints as 818 and 819 are interesting, inasmuch as there are five possible ways of failure.

In 818 we can have failure by—

1. Outer row of rivets failing.
2. Tearing of plates in inner row, and shearing the rivets in the outer ones.
3. Shearing two rivets in double and one in single shear.
4. Shearing one rivet and crushing in front of two.
5. Crushing in front of three rivets.

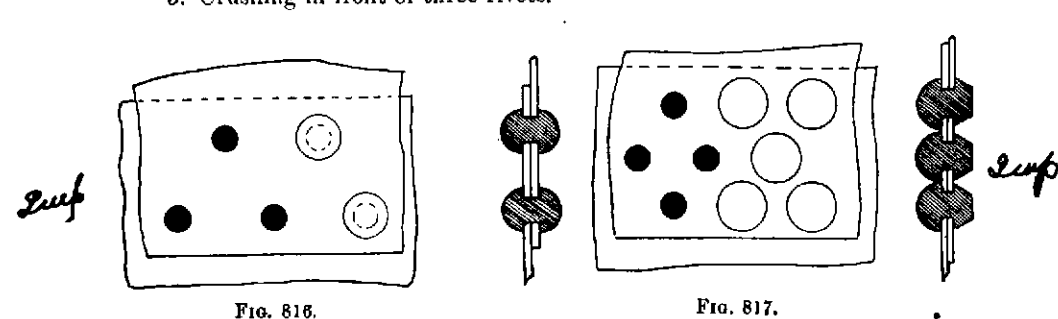


FIG. 816.

FIG. 817.

Likewise in fig. 819 we may produce failure by—

1. Outer row of rivets failing.
2. Shearing at the middle and shearing one of the outer holes.
3. Shearing four rivets in double and one in single shear.
4. Crushing in front of four rivets and shearing one rivet.
5. Crushing five rivets.

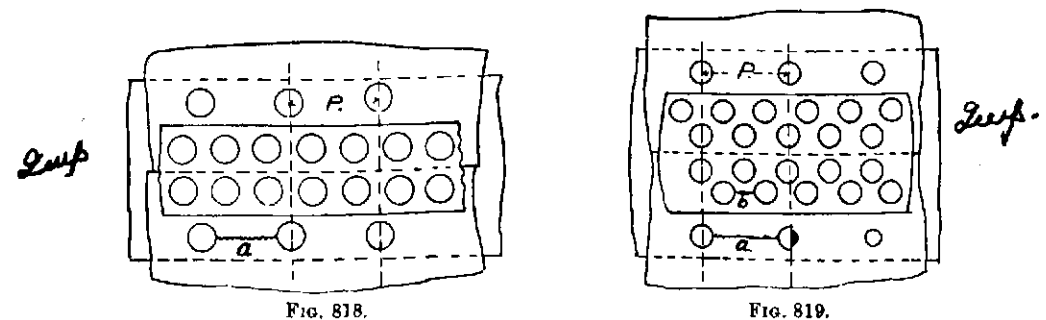


FIG. 818.

FIG. 819.

We have now dwelt somewhat long on the subject of riveted work. The reason for doing so is that upon the proper carrying out of this work so much depends, and it is usually the duty of assistant engineers to be on the works a large part of the time to see that such work actually is properly carried out; hence the reason for our deeming it necessary to explain the work minutely.

We proceed to investigate the behaviour of bolts as a medium for fastening members of machines and structures.

The Strength of Bolts.—A consideration of the behaviour of ordinary screwed bolts under various conditions of loading shows that the selection of

suitable bolts for a given purpose is not quite so simple as one is apt to imagine. First, as to bolts in tension. It is found that bolts subjected to a statical or gradually applied load have a greater breaking strength than one would obtain by merely considering the section of minimum area—i.e. the section at the bottom of the threads. The following analogous case will serve to explain why this is so.

Let two bars of the same length, of the same material, be taken, one of which is turned to a uniform diameter throughout its length, while upon the other bar a number of narrow grooves close together are cut (thus approximating somewhat to a square-threaded screw), the diameter at the bottom of the grooves being the same as the diameter of the uniform bar. If both be now tested to destruction in tension, it will be found that the breaking load for the uniform bar is less than that of the grooved bar, though the minimum area of the two bars is the same. Fig. 819A shows the general shape of the stress-strain diagram that would be obtained from the parallel bar, assuming it to be of good wrought iron or mild steel. It will be noticed that the load increases continually up to some point A, but after this point has been

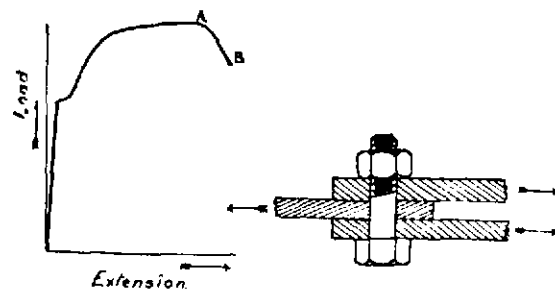


FIG. 819A.

FIG. 819B.

attained the extension continues to increase, though the load is being diminished as shown by the fall of the curve from A to B. This continued extension under a diminishing load is due to a rapid reduction in area caused by a local necking-down of the specimen, which necking-down is followed almost immediately by fracture. The outstanding collars on the second bar have the effect of hindering this necking-down and consequent reduction in area, and this accounts for their increasing the breaking load of the bar.

The thread on bolts is very similar to the collars in the above example, and it has been found that the threads increase the tensile strength of the bolt (always remembering that we are considering steadily applied loads) by from 10 to 19 per cent. Unwin has shown (*Machine Design*, vol. i. p. 188) that, due to the frictional resistances of the thread, etc., the stress in a bolt when it is tightened up against the load is about 17 per cent. greater than the stress due to the load alone. For this case, then, we may roughly regard the extra stress produced by the twisting effect of tightening up as balanced by the increased strength of the section at the bottom of the threads, due to the lesser stressed metal in the neighbouring threads. In the case of bolts subjected to jars or shocks it has been noticed that when the thread is a minus thread—that is, cut into the body of the bolt in the usual way—there is a tendency for the bolt to break off short across the threads, and that the

resistance of bolts to such loads is increased by reducing the sectional area of the shank to that of the base of the threads.

Owing to the greater area of the body of the bolt, the stress per square inch there is less than that at the root of the threads, and consequently the extension per unit length of the body of the bolt is less than that of the threaded section. This causes the extension due to the load to be more or less localised in the threaded portion. If the difference in areas be sufficiently great, it is quite possible that the threaded portion may be strained to breaking point, while the stress in the shank of the bolt has scarcely exceeded the elastic limit. By reducing the area of the body of the bolt, the extension is evenly distributed throughout its length, and a much greater extension than before can be endured without fracture. The resilience or work done in deforming the bar up to the elastic limit is much increased, as also is its flexibility. There are various ways in use for reducing the area of the shank. The bolt may be turned down to the diameter at the bottom of the threads, flats of suitable dimensions may be milled or forged upon it (well rounded, since square corners would introduce another element of weakness), or a further method is to drill a hole down the centre of the bolt, from the head to just above the end of the thread. This last method has the advantage that the torsional strength of the bolt is but little impaired. It should be remembered also in this connection that bolts subjected to a live load must have twice the strength necessary in bolts which have to sustain a slowly applied load of the same intensity. Another point liable to be overlooked is that when bolts are subjected to a shearing stress, as in fastenings, etc., the maximum stress is not given by the ordinary formula—load divided by area.

Consider the case shown in fig. 819B. It is clear that there is a tendency for the pin to bend, and bending will take place unless the frictional resistances at the surface of the bolt are sufficiently great to resist the bending action.

In riveted joints the friction between rivets and plates is sufficient to prevent any bending, and we are therefore justified in calculating the areas required by the ordinary formula given above; but when bolts are used, bending must be allowed for by taking the allowable intensity of stress as three-fourths of that for pure shear.

We have now to consider structural design, and shall start by considering steel stanchions. In Chapter IX. reference was made to cast-iron pillars and formulæ given whereby the safe loads and dimensions of them could be accurately determined. As already stated, their use is rapidly diminishing, single-rolled joists being much used in preference, and, in the case of heavy loads, built-up stanchions. Now we must consider three main varieties of stanchion loading. First of all, in fig. 820 we have an axial load with both ends of the stanchion free to turn about an axis. For a particular load, let L = safe length. Then in fig. 821 we have one end free and one end fixed, and the limiting length is now $1\frac{1}{2}L$, while in fig. 822, with both ends fixed, this value will become $2L$, but, it will be observed, the crippling is confined in each case to a length = L .

Now any stanchion formula will be based on two things:—

1. Load per unit area of the cross section.
2. The ratio which the length bears to the least radius of gyration, which

if I = moment of inertia = $\frac{\sqrt{I}}{A}$ where A = area of section.

It is usually spoken of as the ratio $\frac{L}{\gamma}$, or simply $\frac{L}{\gamma}$.

The usual formulæ employed in connection with stanchion design are those deduced from Euler's experiments, and generally known as Euler's formulæ. They are generally accepted by the engineering profession as the most reliable (except perhaps the Rankine-Gordon).

Let W_b = total buckling load of stanchion and W_s the safe load on the stanchion.

E = Young's modulus of elasticity.

I = moment of inertia of section.

γ = radius of gyration (least).

Then
$$W_b = \frac{\pi^2 EI}{L^2} \quad (438)$$

and
$$W_s = \frac{\pi^2 E \gamma^2}{L^2} \quad (439)$$

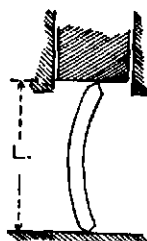


FIG. 820.

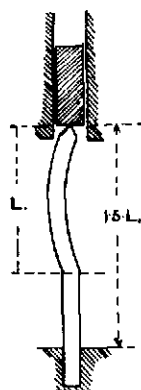


FIG. 821.

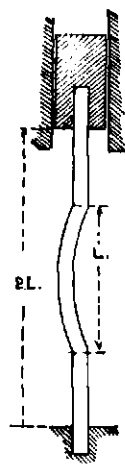


FIG. 822.

W_s will not give the actual safe load, but the safe load per sq. in. of section. In the same way W_b refers to sq. ins. of cross section likewise. The values given are in pounds. E should also be in pounds. Tables are published in any engineering pocket-book giving values of I and γ for any section. E will also be found therein, but for steel in construction work is usually taken as 30,000,000 lbs. per sq. in. Curves have also been prepared, which with a ratio of $\frac{L}{\gamma}$ will show at a glance the safe working loads per sq. in. of cross section. In Formulæ 438 and 439 L does not refer to the actual length of the stanchion, but to the value of L between the points of contraflexure shown in figs. 820, 821, and 822. Now, except for very small loads, a single-rolled joist will usually not be suitable; and even if so, may not be the most economical procedure. The success of stanchion design depends to a great extent upon the proper and scientific distribution of the material at the designer's hands, effected by proper spacing and bracing of the joists. This may be effected in a variety of ways. A common way is that by means

of flange plates in fig. 823. They would be made about 12 in. to 15 in. deep and about 3 ft. to 4 ft. apart, and made of $\frac{1}{2}$ -in. plates. In place of this method, however, which has not a very wide application, the method of fig. 824 may be adopted, a simple system of diagonal bracing. Before going into further detail, however, it will be necessary to see what is the most economical way of distributing our material. Looking up a table of properties of steel joists, of which there are many published (but of them all, the writer is disposed to say that Messrs Dorman, Long & Co. of Middlesborough have produced the most concise), we find that there are two moments of inertia for

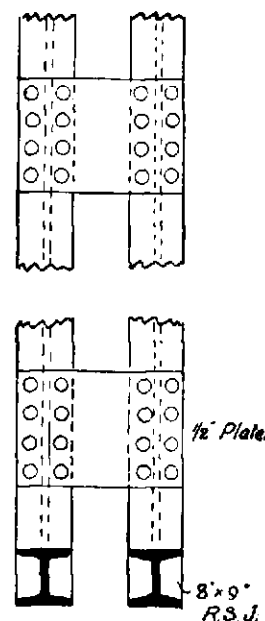


FIG. 823.

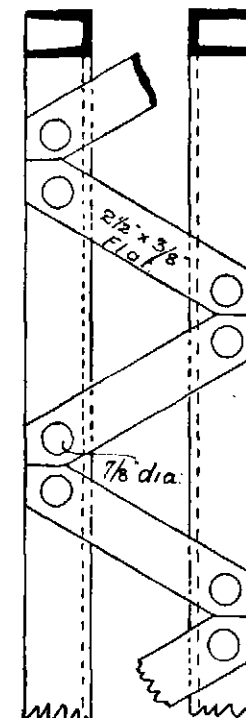


FIG. 824.

any rolled steel joist or compound girder. Referring to fig. 825, we find one taken about xx and the other about yy .

Let I_x = that about xx , and I_y that about yy , in our calculations. Then say we have to provide in a certain stanchion so many sq. ins. of material. Say also we decide to dispose of it between two rolled steel I beams. What is the most economical distance between them? Call it D , and knowing A the sectional area required in sq. ins.,

$$D = 2 \sqrt{\frac{I_y - I_x}{A}} \quad (440)$$

and if we have deemed it prudent to use three joists we have the formula—

$$D_1 = \sqrt{\frac{3(I_y - I_x)}{2A}} \quad (441)$$

When using I beams this formulæ may be applied to distance between the two webs, but in the case of a stanchion consisting of two channel irons back to back, as in fig. 826, the distance D_c must be taken as the distance between the centres of gravity of the sections, a property which will be found in the

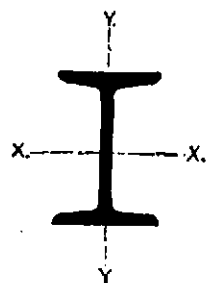


FIG. 825.

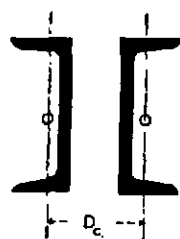


FIG. 826.



FIG. 827.

tables referred to. This is an important consideration. Diagrams showing various common types of stanchions built up of I beams, channels, and zeds are shown in figs. 827-833. As stated before, 827 has not a very wide applica-



FIG. 828.



FIG. 829.

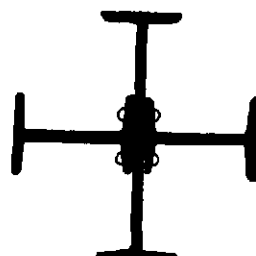


FIG. 830.

tion, nor has 833, which is somewhat expensive, and only suited to light loads.

The flange plates now demand attention. Referring to fig. 834, having spaced the joists according to our previous reasoning, we find that there

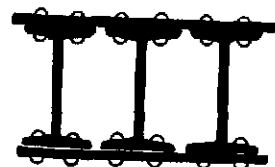


FIG. 831.



FIG. 832.

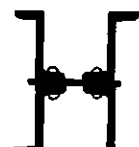


FIG. 833.

is a certain width of flange plate beyond which they are not of economical construction, which is to say, the limiting value of

$$b = 3.4L \quad (442)$$

Hence if we have settled the sectional area of the flange plates, knowing b , the required thickness is easily found. Again, it is usual for steel makers to

roll plates for this purpose 24 in. wide, which will practically limit L to 7 in. or so in economical construction. Now all the previous designs of stanchions will find place in ordinary building operations where the loads and heights are moderate, and the loads themselves usually stationary or "dead." For conditions of great loads and heights much more elaborate construction is necessary, such as the case of a big engineering workshop, with overhead travelling cranes, etc. Figs. 835, 836, and 837 show a type of moderately heavy stanchion, the section being taken about the line AB. Figs. 838, 839, and 840 show a somewhat heavier type, while figs. 841 and 842 show a very large design. In such work as this, in addition to the ordinary side bracing, it is necessary to use also a system of horizontal framed bracing. The shaded areas in places denote packing-pieces. The skeleton diagrams of the three foregoing designs are given in figs. 843, 844, and 845. The dimensions LL are figured on.

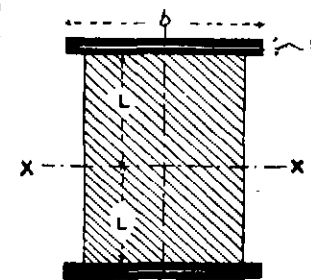


FIG. 834.

Now when using flange plates it is essential that they do not overlap the joists to which they are attached more than $2\frac{1}{2}$ in. on either side. Having then decided to use flange plates, we must find the radius of gyration of them before we can proceed with the design, because our calculations will be altered. We then proceed to find the radius of gyration for the whole section, including the plates.

If γ_1 = radius of gyration of the joists,
If γ_2 = " " " plates,
Then γ = " " " whole stanchion is equal to

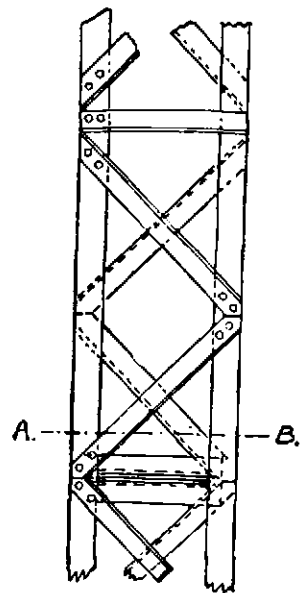
$$\gamma = \sqrt{\frac{\gamma_1^2 + \gamma_2^2}{2}} \quad (443)$$

When working out these calculations, it will sometimes be found that the values of Formulæ 440 and 441 are inconvenient, inasmuch as the value of D will prove less than the depth of two flanges. A fresh approximation would then have to be chosen. Referring again to figs. 843, 844, and 845, for 844 and 845 we have certain values of Q shown. If L = radius of gyration (not strictly accurate but very near), the correct value of

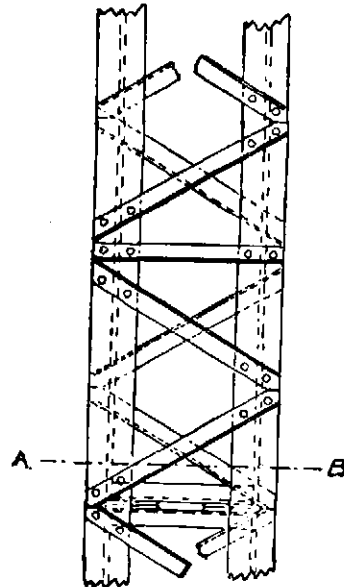
$$Q = \sqrt{L^2 - \left(\frac{I_x - I_y}{A}\right)} \quad (444)$$

A being the sectional area of each single joist as before. Now this formula is not strictly accurate for the types of stanchion shown in figs. 844 and 845, but is very nearly so. The type shown in fig. 843 is a very adaptable one for the purposes of bracing, especially so when the section is made as nearly square as possible; but owing to the conditions imposed by Equation 444, if 844 and 845 are made square on plan, they will not satisfy the conditions of being equally strong in both directions, and the greater the difference between the values of I_x and I_y in the sections used, the more will the ideal built-up design be removed from the square. Hence judgment is necessary. Type 843 is suited to anything up to 160 tons dead load, while 844 and 845 may be used for loads up to 400 tons when designed accordingly. So much for

the actual calculations in reference to stanchion design. The actual formulæ employed are few, because when once the salient points are fixed it is simply a matter of arbitration for the designer to settle the details. Now the



FIGS. 835 and 836.



FIGS. 838 and 839.

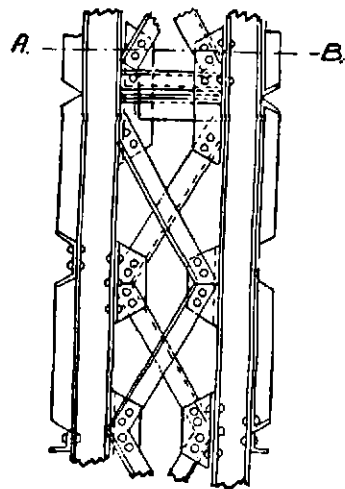


FIG. 837.

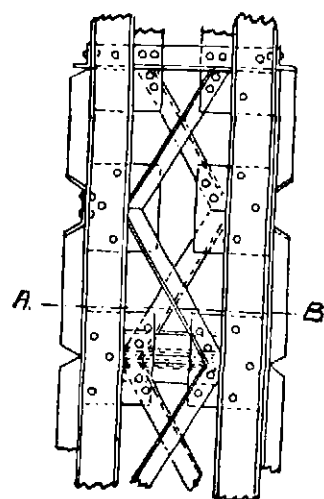
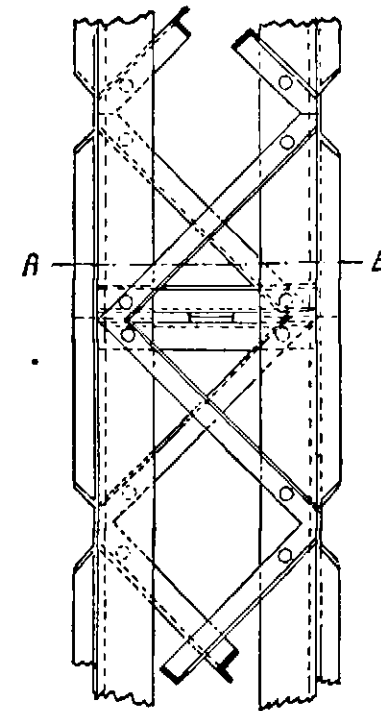


FIG. 840.



FIGS. 841 and 842.

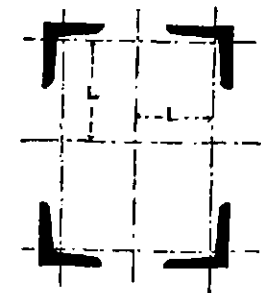


FIG. 843.

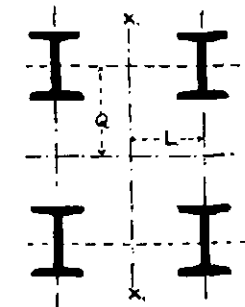


FIG. 844.

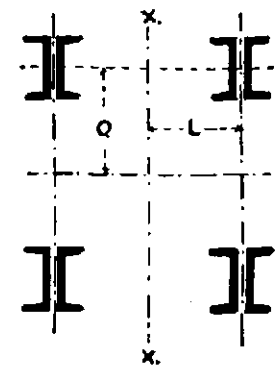


FIG. 845.

purpose of a stanchion is to transmit a load to a firm foundation, and, in an either direct or indirect manner, this load will eventually be transmitted to the ground. If the reader refers to what we said in Chapter XIV. upon foundations, he will see that the actual foundation of a heavy stanchion demands

some consideration, seeing that there is a limiting bearing power to the soil on which such a foundation is going to be placed. Hence he will see that a considerable spreading out of the base of the stanchion will be inevitable, and we are going to show a few of the common and economical means whereby this is effected. The first consideration is the bearing power of the soil. For the various conditions we refer the reader to the chapter mentioned, and shall not make more remarks on the subject, but shall assume that he has a

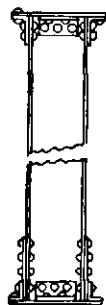


FIG. 846.



FIG. 847.

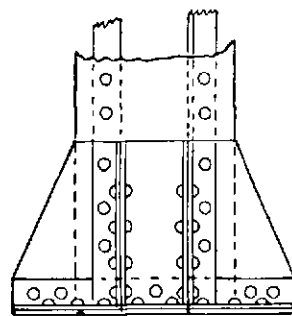


FIG. 848.

certain quality of soil whose bearing power has been accurately determined, and a certain maximum known load on the stanchion. It is only then a matter of practical example and design whereby a means is obtained of evenly distributing a load over a certain area. Let us take a very simple case of a single-rolled joist. It is shown in figs. 846 and 847. The base plate would be

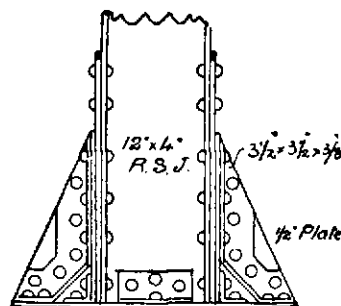


FIG. 849.

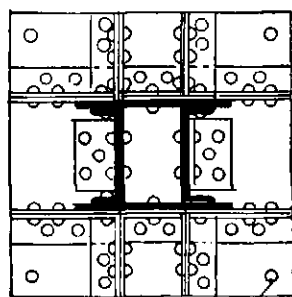


FIG. 850.

bolted by a suitable form of anchor bolt to a concrete base of requisite thickness and superficial area.

Figs. 848, 849, and 850 show details of a foundation for a heavier stanchion, and figs. 851, 852, and 853 show a very large piece of work. It is essential that all side angles in this class of work be cut away on the splay. Base plates are not usually made thicker than $\frac{3}{4}$ in. nor thinner than $\frac{5}{8}$, $\frac{3}{4}$ in. being usually adopted, while the side plates used are generally $\frac{1}{2}$ in. For side angles $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. or 4 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in. are deemed the most serviceable,

while the angle-iron cleats vary from 4 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in. to 6 in. $\times$ 6 in. $\times$ $\frac{1}{2}$ in. according to circumstances. Single or double riveting may be used.

Numerous other types could be given and designed. We have confined our remarks to a few typical examples, leaving the reader to work out fresh designs for himself upon the conditions already set forth. There are, however, four main considerations which may be here enumerated, as follows:—

1. The bearing capacity of the foundations (already discussed).
2. In stanchion design we always consider the lower end (or base) as "fixed" in our calculations (*vide* Chapter IX.); hence, apart from any other considerations whatsoever, the base must be so designed as to provide sufficient area to resist any tilting effect.
3. The lower side of such base plates must be quite flat and strictly perpendicular to the axis of the shaft in all cases.
4. The fastenings between the base plate and shaft must be such that the load is transmitted equally over the whole of that base plate.

In soft ground, where the base plate would ordinarily assume very

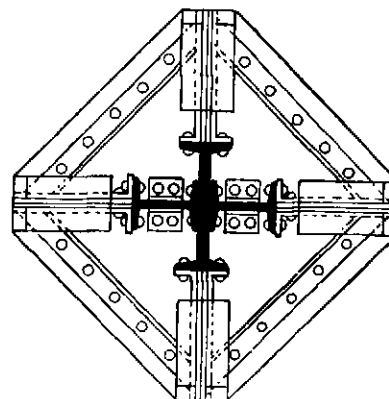
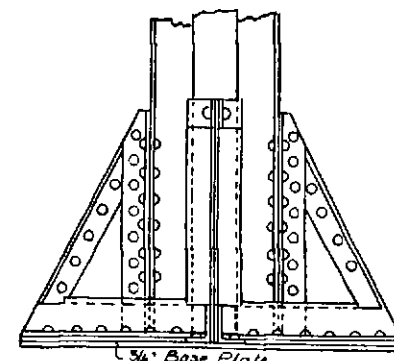


FIG. 851.



FIGS. 852 and 853.

large proportions, the system of grillage foundations is usually resorted to, of which a characteristic type is given in figs. 854 and 855, distance-pieces being provided to keep the joists apart at the proper distance.

The necessary calculations are as follows:—

Let W = total load on the base = load on stanchion + approximate load of the actual stanchion. Let N = the number of joists in one tier of the grillage. Obviously the load on each joist

$$= \frac{W}{N} \quad (445)$$

In the diagram we have two dimensions x and y . Therefore we must take the span of each joist equal to $L - x$ or $B - y$, as the case may be, and then settle upon a value for the size of the joists in question, having load W over a span $= L - x$ or $B - y$. The actual building up of such grillages demands the greatest care. Every part must be flushed up with concrete. Another very common error, due to inferior workmanship, is not screwing up bolts accurately. Inspection of rolled joists will reveal that the inside faces of the flanges are not perpendicular to the web. The reason for this is that if they were so the rolling would be impossible. It may here be stated that when a joist does look very square in this direction, it should be suspiciously looked upon as a foreign joist. All good British makers, however, stamp their names on their material, and this practice will remove all doubt. However, to return to the bolt question. Obviously the use of a bolt without a washer,

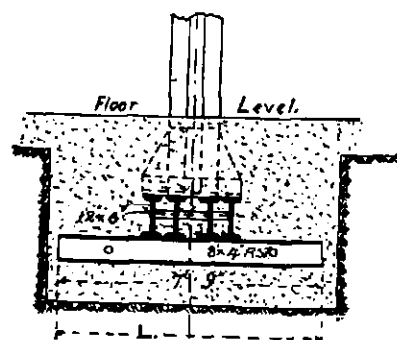


Fig. 854.

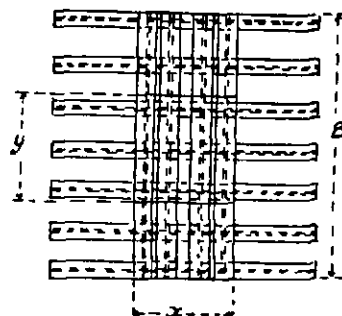


Fig. 855.

or only a flat one, will result in the work not being true. Taper washers must always be used under such circumstances as shown in fig. 856.

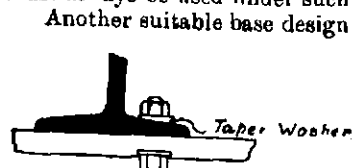


Fig. 856.

Another suitable base design is that shown in figs. 857 and 858, using four bars. In all such cases the number of anchoring bolts may be put to calculation. In order to do so the overthrowing moment is required; we then get the correct moment of stability by suitably proportioning the depth and number of proposed bolts to enclose a sufficient weight of concrete. The connection between such bolts is usually made by flat bars, but where extra strength is required channel bars (—) are used; in any case, if the foundation is brick instead of concrete, channels 3 in. deep must be used, so as not to interfere with the courses of the brickwork. It is also a great advantage to flush up a stanchion base with concrete right up to where it begins to spread out for the foundation. Such a procedure tends to make the stanchion more fixed by reducing any sliding tendency to a minimum. When carrying out the actual construction, stanchion bases are usually made about $\frac{1}{2}$ in. lower than the actual required finished level. The stanchions are then placed thereon, and raised and plumbed up by means of small *hard steel* wedges. They are then finally grouted up with neat cement. The anchoring bolts are treated

in the same way as described in a previous chapter under Engine Foundations. The boxes, to be easy of withdrawal, must be coated with tallow.

Another variety of grillage is given in fig. 859. When channel bars are used for holding the anchor bolts, they may be either laid up or down. Those laid with the channel up only need flushing with cement, but those in which the channel is turned down require holes in them, so as to allow neat cement being poured in in a liquid state. What has been said in reference to foundation or base plates may also be applied to the cap plates, but these latter are not usually so large. It is most essential that when stanchions

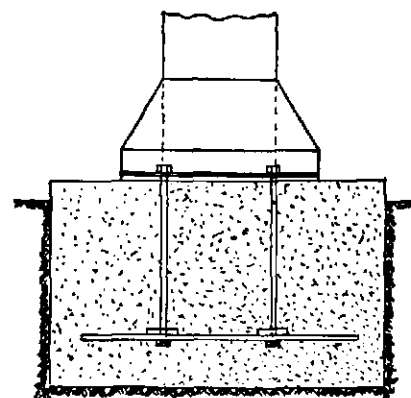


Fig. 857.

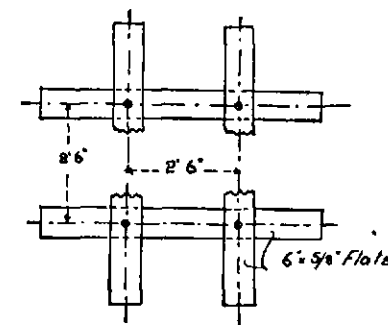
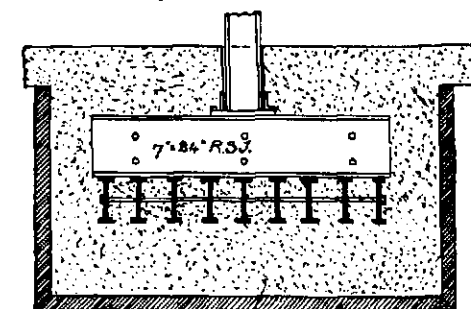


Fig. 858.

are designed as we have been advising, the load be absolutely axial, because when we come to consider eccentric loading we have different stresses to deal with. Such cases are not, however, very usual in constructional work, except the cases of overhead travelling cranes supported on steel stanchions. The way that such a load would be transferred to a stanchion is shown by fig. 860. A simple bracket for this purpose should never be allowed. Sometimes, in very long stanchions, splicing has to be resorted to. It is not a good practice, and is really better avoided if possible, but such is not always so. However, a fairly common case is that shown in figs. 861 and 862, as applied to single-rolled steel joists. Figs. 863, 864, and 865 show further details as applied to a built-up stanchion, and a still more complicated case in figs. 866 and 867. Other cases will readily present themselves to the reader. All butt ends should bear truly upon one another, and be faced in a slotting or planing machine, truly square. All rivet-holes must be very carefully drilled, and no bolts used in this particular work. In the case of a high building, a change in the scantling of the stanchion will be prudent, with a view to subsequent

Concrete. Brickwork.
Fig. 859.

economy. In such cases, where possible, it is advisable to make the reduction in the web of the joist rather than the flanges, using a plate sandwiched between the two joists and angle-cleat connections, instead of having two planed faces abutting. The bearing plate, however, must be faced

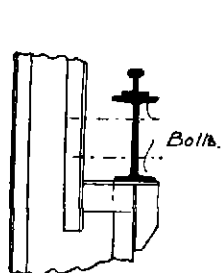


FIG. 860.

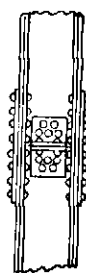


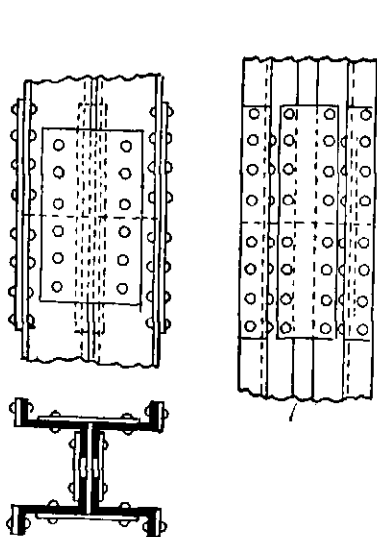
FIG. 861.



FIG. 862.

up true, and be about $\frac{3}{4}$ -in. thick. The type of stanchion shown in fig. 843 will often require splicing. It must be done between two pairs of bracing bars.

The fixing of joists and girders to stanchions at intervals between the top and bottom next demands attention. It is a practice which, although in many cases unavoidable, is not to be advised, except where two such



FIGS. 863, 864, and 865.

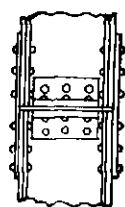


FIG. 866.



FIG. 867.

cases, dealing with equal loads, appear on opposite sides of the same stanchion and at the same level. The fixing is effected by means of brackets, the design of which requires a little care. Figs. 868 and 869 show the details of such a case applied to a single-rolled steel joist. Figs. 870, 871, and 872 another similar case, while a large detail is presented in figs. 873, 874, and 875. Other means of effecting the same end are shown by figs. 876 and 877, in which

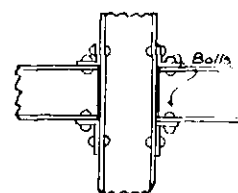


FIG. 868.



FIG. 869.

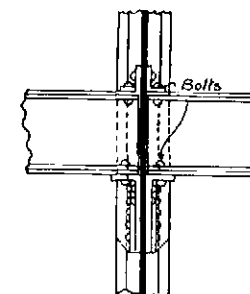


FIG. 870.

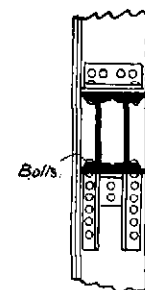


FIG. 871.

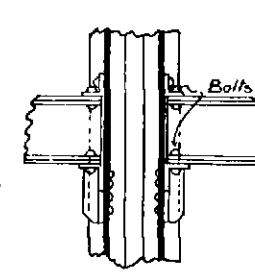


FIG. 873.

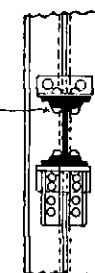


FIG. 874.

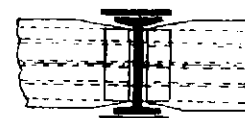


FIG. 872.

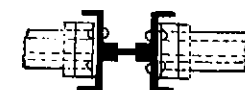


FIG. 875.

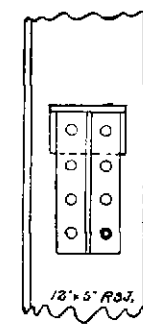


FIG. 876.

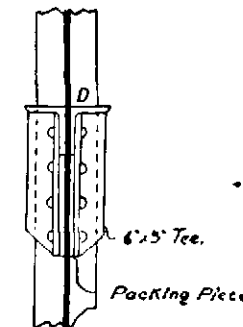


FIG. 877.

do not at present concern us. But now, brace them by a system of diagonal bracing as in 884, and see what an excessive load P will do, as in fig. 885. In fact, fig. 884 shows a very effective form of bracing. The diagonal braces should, however, if possible be at 45° to the stanchions; and if the height of the structure prevents this condition being fulfilled, then the bracing should be divided into bays. When, however, a clear headway is required for some purpose, we resort to knee bracing, as in fig. 886. The knees should always be at 45° likewise, but the very important point is the stanchion anchorage, which must be specially firm and well designed. Knees placed right up in the corners are no use; they must come well down, as far, in fact, as headroom

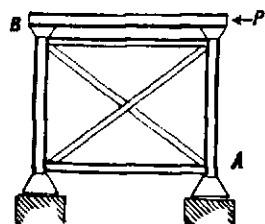


FIG. 884.

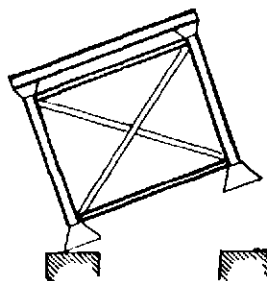


FIG. 885.

will permit. Now in the case shown in figs. 884 and 885, if there was only a force due to wind to consider $= P$, one brace AB would fulfil the conditions of stability. The other one, however, is usually put in, and constitutes a *redundant* member (Chapter XII.). A very common way to effect a joint where two bracing-bars cross each other is by a forged steel ring in fig. 887;

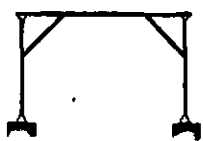


FIG. 886.

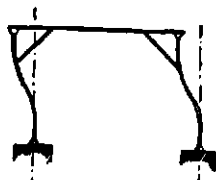


FIG. 886A.

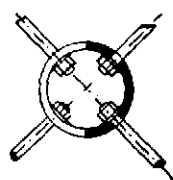


FIG. 887.

this puts all the bars into tension. Now, when considering the horizontal force P setting up stresses in a system of stanchions, which is in turn transferred to those stanchions by the bracing, we can consider a certain vertical component due to the force P , which we can add to the total load on the stanchion already under consideration. For instance, take fig. 888.

Let V be the value of this added load.

Then
$$V = P \frac{x}{y} \quad (448)$$

while the tension in the tie rod

$$= T = P \times \frac{\sqrt{x^2 - y^2}}{y} \quad (449)$$

while AB has a compression $= P$. In BC, however, the load acts upwards, and if it is greater than the actual vertical load, will cause failure as in fig. 885.

Let us consider a row of diagonally braced stanchions, having a horizontal force P at one end (due to wind, say). We then have a series of stresses very like a braced girder (Chapter X.), each panel of the bracing taking an equal share of the load. Hence each stanchion will have an equal value of V , with the exception of the windward one.

Now say we have N stanchions. Obviously we have $N - 1$ braces in action together.

Each length of the top girder should be capable of standing the maximum value of P . Such a system under load is shown in fig. 889. Obviously the first girder will transmit a total force P . At B the force becomes

$$P - \frac{P}{(N - 1)} \quad (450)$$

and so on right to the end, where only

$$\frac{P}{(N - 1)} \quad (451)$$

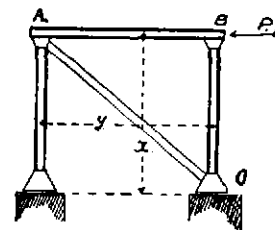


FIG. 888.

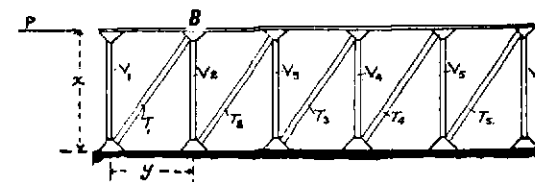


FIG. 889.

will remain. If we had struts at the base also, they would be in horizontal shear, equal in value to Formula 451. In the figure no base struts are employed, but this shear will still exist. The extreme leeward stanchion is, however, free. Now we want to resolve the components of the wind pressure to the vertical. At any member (counting from the end at which P acts) N ,

$$V_N = \frac{Px}{y(N - 1)} \quad (452)$$

V_N being the amount of vertical load to be added to that member

While the stresses in the diagonals (always compressive under these conditions)

$$= T_N = \frac{P \sqrt{x^2 + y^2}}{y(N - 1)} \quad (453)$$

T_N being the only force in the diagonals, will fix their sectional area. Now the proper securing of the ends of these diagonals demands attention. A suitable joint is shown in fig. 890, the braces being sprung over each other where they pass. Fig. 890A is for use with the system shown in fig. 887.

packing-pieces are shown. Now where packing-pieces are used, as in this case, they should always be carefully made to abut against the lower edge of the angles, to assist in taking up the shear. With this type, however, the shelf surface is always somewhat small. When dealing with brackets it is necessary to consider carefully the following points:—

1. The bearing stress on the rivets.
2. The compression on the supporting tee or angles. Now the bearing area of any rivet is the product of the diameter of that rivet and the thickness of the thinnest plate it connects to if we have to support a load of W tons by means of a rivet d inches in diameter, the thickness of the thinnest plate being equal to t inches. We proceed to find the required number of rivets having a safe resistance to shear stress of 6 tons per square inch = f .

Then

$$N = \frac{W}{d \cdot t \cdot f} \quad (446)$$

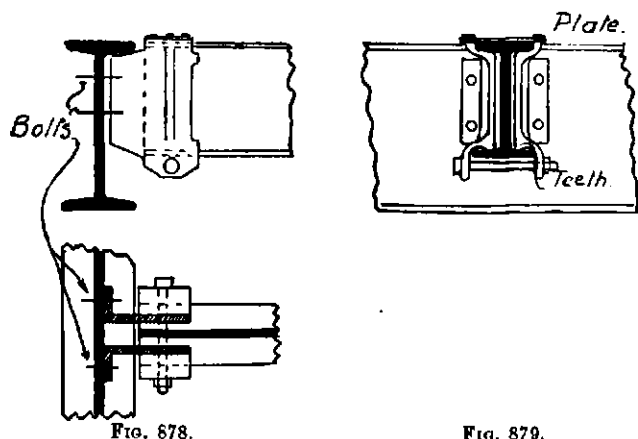


FIG. 878.

FIG. 879.

Or, say, we have the stanchion web = T inches in thickness, the safe load on that particular bracket in tons

$$= \frac{T}{8}(D + t) \quad (447)$$

It is preferable, where possible, to use two angles back to back in cases like these, instead of tees, because they are of necessity made in larger sizes, and so allow D , fig. 877, to be larger.

The shelf angles need not be greater than $\frac{1}{2}$ in. in thickness, the brackets being $\frac{1}{8}$ in. to $\frac{3}{8}$ in., while the actual overhang of the angles may be safely made $\frac{1}{8}$ in. to $\frac{1}{2}$ in. greater than D . In all such work it is advisable to place rivets at 3-in. pitch, starting at $1\frac{1}{2}$ in. from the lower end of the brackets. In place of using built-up brackets, a beam framing clamp has lately been introduced and brought under the writer's notice, and is shown in figs. 878 and 879, which shows the construction clearly. It is very simple and effective, and consists of two main iron castings as shown. These, on the lower side of the lower flanges, have a series of teeth, which bite into the lower flange of the joist to be supported. It allows the beam to have its ends only roughly cut,

and also a square end, and saves much detail. In the writer's opinion, however, the main jaws should be made of cast steel, or if possible drop forgings, in preference to cast iron, which is not advisable in constructional work, especially under transverse load.

Before leaving the actual design of stanchions pure and simple, it would be well to present to the reader yet another type which, when built up, would carry an enormous load—anything up to 1000 tons. It is illustrated in fig. 880. How such a stanchion would be actually built up may puzzle the reader

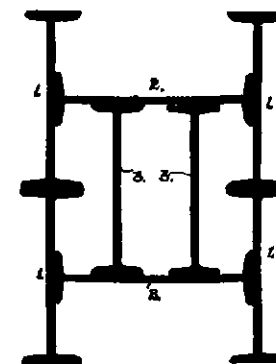


FIG. 880.

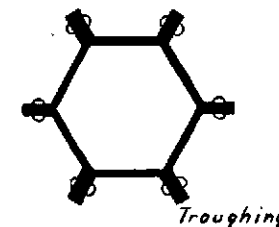


FIG. 881.

at first. First of all, the joists 2-2 are riveted to the joists 1-1. 3-3 are then brought up and riveted to 2-2 after having their flanges joined up.

A very favourite type amongst architects, who for artistic reasons require the pillar to be afterwards cased in, in circular form, is that shown in fig. 881. It is built up of troughing irons. It may then be cased in with expanded metal lathing and the plaster fixed thereon.

We proceed to examine the bracing of stanchions. Rows of stanchions,

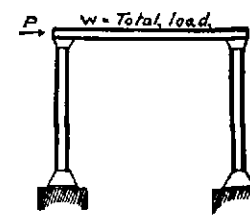


FIG. 882.

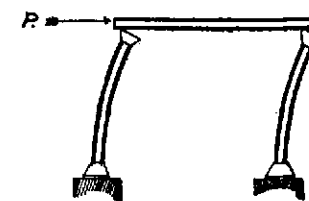


FIG. 883.

especially in big buildings where their duty is mainly to support roofs or travelling cranes, such as occurs in large factories, power-houses, warehouses, and similar cases, may be greatly assisted in their duty by a suitable system of bracing, thereby perhaps reducing their sectional area, and altogether facilitating the erection of the building and making a better job. The bracing would also be of service in other ways, as will be pointed out. Bracing, however (in fact like all bracing we shall consider), is apt to act at times both as a strut and a tie. This calls for some care in the design. Take fig. 882, two ordinary stanchions and a girder, and a force P . Failure will, under ordinary circumstances, occur as in fig. 883, or less likely in other ways which

A much more elaborate system of bracing is shown in fig. 891 (diagram only).

The correct formulæ to use in design are

$$V_1 = \frac{P_1 - x_1}{(N-1)y} \quad (454)$$

and

$$T_1 = \frac{P_1 x_1^2 + y^2}{(N-1)y} \quad (455)$$

The other values may be taken accordingly.

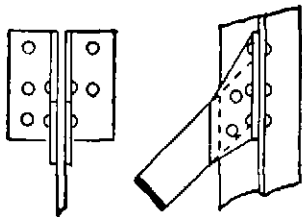


FIG. 890.

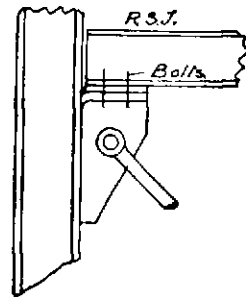


FIG. 890A.

It will be unnecessary to worry about the loads on the intermediate stanchions. They should all be designed for the maximum load on the extreme leeward one.

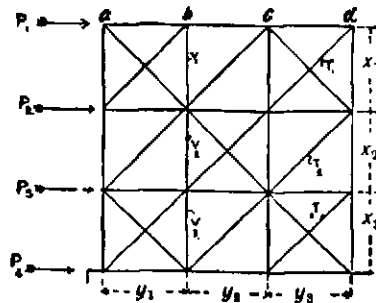


FIG. 891.

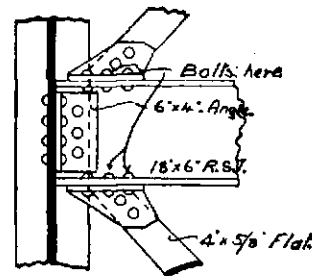


FIG. 892.

Going downwards, we shall see that V_2 is greater than V_1 , and V_3 than V_2 , and so on.

Thus

$$V_2 = \frac{(P_1 + P_2)x_2}{y(N-1)} \quad (456)$$

each time adding an extra value of P . In the same way T will increase.

The shear at the foot of each stanchion

$$-S = \frac{P_1 + P_2 + P_3}{(N-1)} \quad (457)$$

and if we have base struts, their value of

$$S = \frac{P_1 + P_2 + P_3 + P_4}{(N-1)} \quad (458)$$

In order, however, to prevent a lifting action, we must have a load on the windward stanchion = $V_1 + V_2 + V_3$, etc. The rivets at the end of the diagonals must safely resist a shear = V . The correct joint for such a case as fig. 891

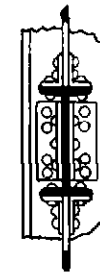


FIG. 893.

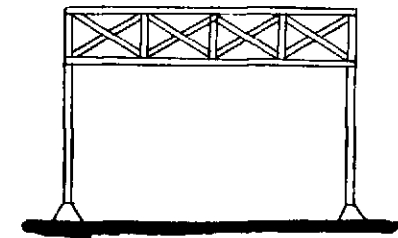


FIG. 894.

is shown in figs. 892 and 893, while the actual sections of the bracing may be one flat, two flats with packing, two flats with tubular distance-pieces, bulged or not bulged, or two tees back to back, as the case may be. Now, owing to the strain in the diagonals due to load, we have an induced B.M. in the

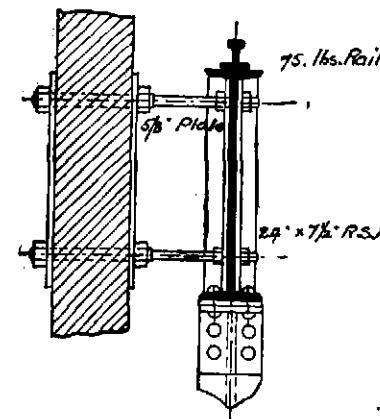


FIG. 895.

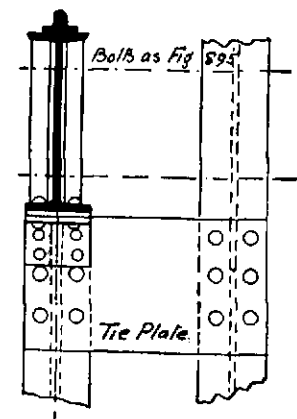


FIG. 896.

stanchions, whose maximum value will be at the base. Let the strain = D and let Z = the modulus of the proposed section.

Then the extreme fibre stress set up

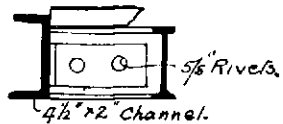
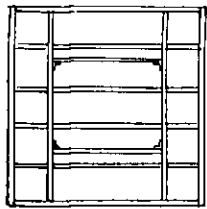
$$= f = \frac{WD}{Z} \quad (459)$$

W being the vertical load, to which f must be added. In fig. 892 it is essential to use bolts for securing the diagonals to the horizontals.

The reader has no doubt heard of a portal bracing. Fig. 894 shows one. To be economical, they need to be very deep. Now, when the load P comes on, we shall have a deflection, like fig. 886A, when the base is properly anchored (this is much exaggerated). We shall proceed to give a practical example of stanchion design before closing our chapter, but there are two types of runways for an overhead crane, which have been suggested to the writer by an engineering friend since the previous example was considered. They are quite simple and self-explanatory, and are shown in figs. 895 and 896.

Let us now take the following example:—

It is required to support a tank of square shape as shown in fig. 897, and by working out the load we find we can take it at 1 cwt. per sq. ft. Then we shall provide two main stringers of $4\frac{1}{2} \times 2 \times \frac{7}{16}$, and cross joists of $3 \times 1\frac{1}{2}$ in. $\times$ 4 lbs., placed 18 in. C—C. Allowing for wind pressure of 40 lbs. per sq. ft., the moment at the base due to it is required. The total weight of tank plus allowance for tower is taken as $6\frac{1}{2}$ tons. Working out the necessary calculations (which we shall not repeat), the B.M. at base = 404 in.-lbs.



Figs. 897 and 898.

This will be resisted in four ways.

1. In compression, by the area of two angles multiplied by the distance to the centre line.
2. In tension likewise.
3. In compression, by the area of two angles multiplied by the full width of the base.
4. In tension, by the area of two angles multiplied by the full width.

Let us consider the latter method for convenience.

Let the area of two angles = 5 sq. in. and depth = 10 ft., then using $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ L irons the stresses produced

$$= \frac{W}{A} \pm \frac{M}{Z} = \frac{6.16}{5.0} \pm \frac{403}{10 \times 12} = 1.23 \pm 3.30$$

= 4.53 tons per sq. in. compression and 2.07 tension.

Let us assume that the angles act as a stanchion with a height of 10 ft., then W safe load, using the following formula,

$$= \frac{F}{1 + \frac{1}{a} \left(\frac{l}{d} \right)^2} \quad (460)$$

(Rankine-Gordon modification)

$$= \frac{7.5}{1 + \frac{1}{1500} \left(\frac{120}{2.5} \right)^2} = 3 \text{ tons per sq. in.}$$

so that our first approximation will be too small.

Let us investigate the formula,

$$W = \frac{FS}{1 + \frac{1}{a} \left(\frac{l}{d} \right)^2} \quad (460a)$$

The formula quoted is that drawn up by Tredgold, but known as Gordon's formula. It takes account of the least width (d) of a strut as one of the elements upon which the strength depends. Experiments, however, do not agree very closely with the results of calculation by this formula, and Rankine altered it by taking into account the least radius of gyration (r) instead of the least diameter or width, and this brought calculation and experiment into close agreement. The $\frac{1}{a}$ in the Gordon formula is made

up of three elements $\frac{1}{a} = \frac{m}{nq}$, in which m = a fixing modulus, say 1 for both ends fixed, 2.5 for one end fixed and one rounded, 4 for both ends rounded, 16 for one end fixed and the other free; n = a shape modulus, say $1\frac{1}{2}$ for hollow cylindrical section, 1 for solid rectangular section, $\frac{3}{4}$ for solid cylindrical section, $\frac{1}{2}$ for cross, L, H or T section, $\frac{1}{3}$ to $\frac{1}{4}$ for built-up beams; q = a strength modulus, say 500 times the tensile working strength in tons per sq. in. Suppose the case to be a hollow cylindrical cast-iron pillar, ends flat and fixed, the ordinary value for $\frac{1}{a}$ is $\frac{1}{800}$; but to show the confusion

that exists, it may be pointed out that for the same thing $\frac{1}{400}$ is given by Hurst, Molesworth, Wray, Campin, Horner, Lineham, Stoney, and *The Builder*; $\frac{1}{600}$ by Trautwine; $\frac{1}{800}$ by Rankine, Fiddler, Cambria Steel Co., Mitchell, and *Notes in Building Construction*; $\frac{3}{800}$ by Moos.

Proceeding, let us try a $4\frac{1}{2}$ -in. $\times$ $4\frac{1}{2}$ -in. $\times$ $\frac{5}{8}$ -in. angle, and then the stresses produced will be equal to

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{6.15}{10.5} \pm \frac{403}{10 \times 12} = .6 \pm 3.30$$

= 3.90 tons compression and 2.70 tons per sq. in. tension, in which case, using Formula 460, the safe permissible stress is—

$$\frac{7.5}{1 + \frac{1}{1500} \left(\frac{120}{2.5} \right)^2} = 3.93 \text{ tons per sq. in.}$$

Therefore this section will do nicely. We have purposely taken a wrong section previously to this in order to show the reader that in such cases as these the first or even the second and third approximations will not give the required results, when the calculations must be gone into again.

The horizontal ties would be 3-in. $\times$ 3-in. $\times$ $\frac{3}{8}$ -in. angles, and the cross braces $2\frac{1}{2} \times \frac{1}{4}$ in. flats. A firm concrete base 14 ft. square $\times$ 4 ft. thick should be provided, and into it the angles should be bedded 2 ft. 6 in. deep. Details of the construction are shown in figs. 897, 898, 899, 900, and 901.

Another interesting case appears in fig. 901A.

It is required to find the dimensions of a suitable column AC. For this case we shall assume the wind pressure at 20 lbs. per sq. ft. We then proceed to resolve the wind pressure into 8 lbs. vertical and 3 lbs. horizontal on the roof itself. The roof bay = 20 ft. $\times$ 43 ft. = 860 sq. ft. and $\times$ 8

= 6880 lbs. = 3 tons as the vertical component of the wind pressure, and $860 \times 3 = 2580$ lbs., say 1 ton horizontal component. We shall consider our column as fixed at A and hinged at C, being loaded at B. We must now obtain the point of contraflexure above ground-level, which is very often

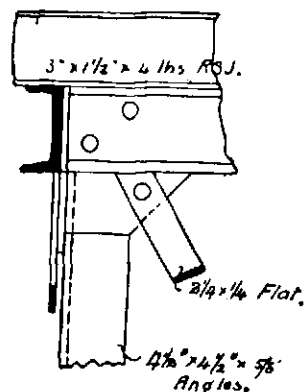


FIG. 899A.

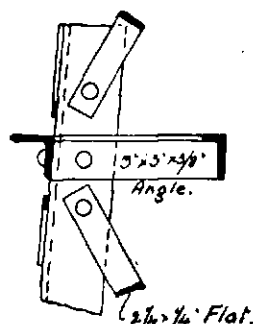


FIG. 899B.

assumed in practice as half AB, but is better found with accuracy by the formula—

$$Y = \frac{V_b(V_b + 2V_a)}{2(2V_b + V_a)} \quad (460b)$$

$$= \frac{26(26 + 78)}{2(52 + 39)} = Y = 14.8 \text{ ft}$$

We then only take into consideration the wind pressure above Y, which is

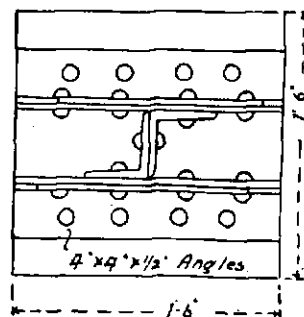


FIG. 900.

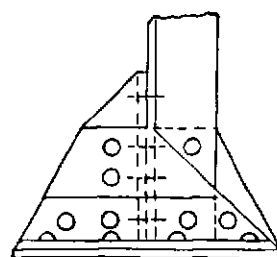


FIG. 901.

equal to $20(39 - 14.8) = 4.4$ tons. So with a roof load of 1 ton horizontal we have equal reactions R_1 and $R_2 = 2.7$ tons each at Y, the point of contraflexure. Now the moments about the foot of the knee braces

$$= R_1 \times V_b - Y \quad (460c)$$

= $2.7 \times 11.2 \times 12 = 363$ in.-tons on the leeward stanchion; that on the

windward one, being somewhat less, is not considered; so taking moments about points of contraflexure and dividing by the span we have—

$$= \frac{1(\text{ton}) \times 32.2}{79} + \frac{4.4 \times 11.2}{79} = 1 \text{ ton.}$$

(Note.— $11.2 = V_b - Y$.)

Now the compression on the column = $\frac{1}{2}$ of the total, leaving $\frac{3}{4}$ of a ton = .75. Therefore the rolled joist which is used must be capable of withstanding a load = $3 + 1 + .75 = 4.75$ tons. So if A = area of our stanchion in sq. in. and L = length also in ins., and if r = its radius of gyration about XX (in ins. likewise), we have—

$$S = \frac{W}{A} \left(1 + \frac{l^2}{cr^2} \right) \quad (460d)$$

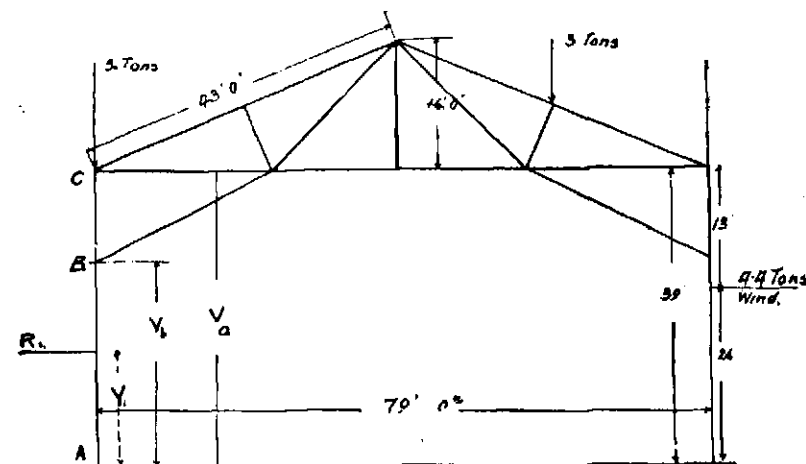


FIG. 901A.

Let us try a 12-in. x by 6-in. x 54 lbs. R.S.J.

$$\therefore S = \frac{4.8}{15.88} \left(1 + \frac{468^2}{24000 \times 4.86^2} \right)$$

$$= .35 \left(1 + \frac{22062}{24000 \times 23.62} \right)$$

$$= .35 \times 1.39 = .475 \text{ ton per sq. in.}$$

the radius of gyration being, of course, taken in the direction of flexure.

Now BM = 363 in.-tons and M = modulus of section = 62.58. $\therefore S_b =$ main comp. stress on the extreme fibres = $\frac{BM}{M} = \frac{363}{62.58} = 5.9$ tons per sq. in., and adding the .475 ton as above = $5.9 + .475 = 6.375$ tons per sq. in., which is quite a safe load for steel.

Designing a Cast-Iron Pillar by the Gordon Formula.—We have already treated the subject of cast-iron pillars in a previous chapter, giving therein certain formulæ whereby they could be designed.

Many, however, prefer to use Gordon's formulæ direct as being more accurate, although they are certainly more troublesome to use. To be able to dispose of the metal in the most economical way in a pillar of this description is not nearly so easy a matter as it first appears, and a little foundry experience is always valuable in this departure. In the first place, to cast a pillar less than $\frac{1}{2}$ in. thick would not be advisable, and the usual conditions on the design of castings previously set forth must be adhered to.

The usual foundry practice is to make the diameter about $\frac{1}{24}$ the height and the thickness $\frac{1}{12}$ the diameter. These rules, however, must not be taken as binding, because this practice will vary greatly. We proceed to investigate a case.

Required—

A cast-iron stanchion (or pillar, as they are usually called to distinguish them from steel stanchions) 15 ft. high for a load of 40 tons.

$$\text{Gordon's formula} \quad W = \frac{f}{1 + c\left(\frac{l^2}{d^2}\right)} A \quad (460e)$$

Transposing to give A we have—

$$A = \frac{W\left(1 + c\frac{l^2}{d^2}\right)}{f}$$

$$\therefore A = \frac{40\left(1 + \frac{1}{320} \times \frac{(15 \times 12)^2}{81}\right)}{6}$$

Investigating our equation, we have the constant = $1/320$, being recognised as correct in most text-books for this particular case. Assuming one end fixed and one end free, we are taking a safe stress of 6 tons, which would be quite suitable for a dead load, while we are taking a 9-in. pillar at random to see what sort of a result it will give.

Continuing—

$$A = \frac{40\left(1 + \left(\frac{1}{320} \times \frac{32400}{81}\right)\right)}{6}$$

$$= \frac{40 \times 2\frac{1}{3}}{6} = \frac{20}{3} \times \frac{9}{4} = 15 \text{ sq. in.}$$

Now a 9-in. pillar $\frac{5}{8}$ in. thick has a total sectional area of $63.6 - 47.1 = 16.50$, a pillar which would do, as it gives a fairly good distribution of metal. It is the duty of the designer, however, to repeat these calculations and see if, by using other diameters and thicknesses which, being within reasonable limits, would give safe results and a lighter pillar, because such castings usually cost so much per cwt.

Fig. 901b shows a suitable design for a C.I. pillar.

Lattice Bracing on Stanchions.—Although this work is not usually subject to more mathematical consideration than is given on the subject in this

chapter, yet the following method of accurately determining the design may be of use to the student. Referring to fig. 901c,

Let l = length between bracing.
 L = length of column.
 r = radius of gyration of each section.
 R = radius of gyration of the whole.

$$\text{en } l = \frac{Lr}{R} \quad (460f)$$

and must not exceed eight times the least width of the segments connected. In practice it is usual to employ—

(1) For 15-in. channels or sections built of $3\frac{1}{2}$ -in. and 4-in. angles, $2\frac{1}{2}$ -in. lattice bars having $\frac{1}{2}$ rivets.

(2) For 12-in. and 10-in. channels or sections built up of 3-in. angles, $2\frac{1}{2}$ -in. lattice bars are used with $\frac{1}{2}$ -in. rivets.

(3) Likewise with 9-in. and 8-in. channels or sections of $2\frac{1}{2}$ -in. angles, 2-in. bars and $\frac{1}{2}$ rivets are used.

But in no case should any

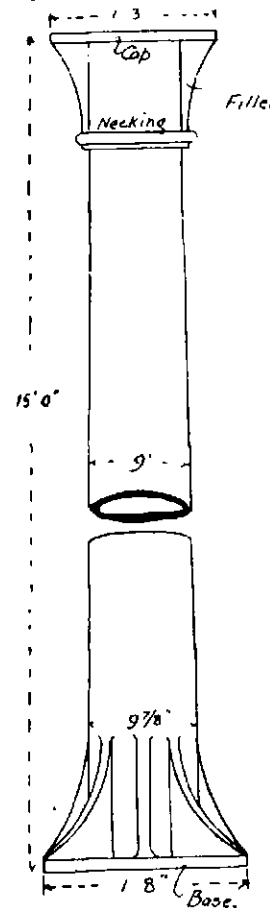


FIG. 901a.

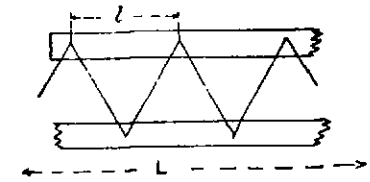
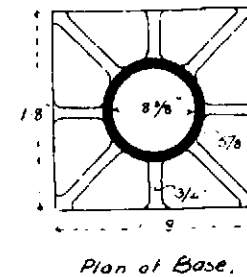


FIG. 901c.

lattice bar when single be thinner than $\frac{1}{16}$, or, if double, than $\frac{1}{80}$ of the distance between the rivets connecting them to the main members, assuming the double bars to be riveted at their intersection.

$$\rho = \sqrt{\frac{I}{A}} \quad (465)$$

$$P = \frac{My}{I} = \frac{M}{Z} \quad (466)$$

and so on, the above being the most useful and commonly occurring modifications.

Let us take a case. A beam has a span of 20 ft. and is to support a wall of a building whose total weight = 10 tons. The weight of roof = say 4 tons and the floor with its load = 6 tons.

$\therefore$ total load = 20 tons.

The load being evenly distributed, the maximum bending moment will be equal to $\frac{WL}{8} = \frac{20 \times 20 \times 12}{8} = 600$ tons-ins. = M . Let us try a 15 in. $\times$ 6 in. I

beam. We find the value of $I = 629$, and $P = 6\frac{1}{2}$, and $y = \frac{15}{2} = 7.5$. $\therefore \frac{600}{629} =$

$\frac{6.5}{7.5} =$ (using Equation 466) $P = \frac{600 \times 7.5}{629} = \frac{4500.0}{629} =$ a little over 7 tons per

sq. in. for extreme fibre stress. Now this is over $6\frac{1}{2}$, but as the particular load is quite steady and is uniformly distributed the beam will be quite safe. This is a very simple case, given for sake of example only. No doubt the reader is aware that in section books which contain tables of the properties of rolled-steel joists each beam has appended to it a certain maximum safe load for a certain span. To use such tables, no doubt, affords a great saving of time, but the necessary competition amongst steelmakers causes the values to be the absolute maximum admissible, and hence great care should be exercised in using them, and for this reason we have given in detail means whereby the proper sections can be arrived at. The particular value of Formula 461 is that it is universally applicable to any shape of beam whatsoever, provided the moment of inertia and y and P are known. The values of I for all manner of shapes are easily got from tables, or we have also shown how to work them out if required.

For convenience, however, we can state the more common values as follows:—

1. For a rectangle with dimensions b and h in inches,

$$I = \frac{bh^3}{12} \quad (467)$$

For a square, side = b inches,

$$I = \frac{b^4}{12} \quad (468)$$

For a hexagon, side = b inches,

$$I = \frac{5b^4\sqrt{3}}{16} \quad (469)$$

For a circle, diameter = d ,

$$I = \frac{\pi d^4}{64} \quad (470)$$

For a tubular (circular) section,

Inside diameter = d_1 ,

Outside „ = d_2 ,

$$I = \frac{\pi}{64} (d_2^4 - d_1^4) \quad (471)$$

CHAPTER XXXVII.

BEAMS AND GIRDERS.

We now proceed to investigate the second main division of constructional steelwork known as beams and girders. We have already dealt with the theory of beams in a previous chapter, as we did with stanchions: it remains in our present chapter to place before the reader some examples whereby the foregoing theory may be successfully put into practice.

It will, however, be necessary to investigate some further rules for determining safe loads, skin stresses, etc., in addition to those already given.

First of all we have the very familiar equation—

$$\frac{M}{I} = \frac{P}{y} \quad (461)$$

where M = the greatest bending moment on the beam in tons-inches. This will be found by the various means suggested in Chapter VIII. and which will not be here repeated.

I = greatest moment of inertia of any section about the neutral axis passing through the centre of gravity of that section.

y = distance in inches of the outermost fibre from the neutral axis (as fully explained in the chapter referred to).

P = safe skin stress in tons per square inch, which may be taken as $6\frac{1}{2}$, because this figure is recognised by the Board of Trade.

Now let Z = section modulus.

Every beam has a section modulus. They are usually given with accuracy in tables of standard beams. When a beam section, however, is not symmetrical there will be two values of Z . For practical use, however, only one value is generally given.

The sectional area of the section is usually denoted by A square inches.

Any other moment of inertia parallel to I is called I_p and distance between them V .

The radius of gyration = ρ .

Then from the fundamental rule (461) which applies to any beam of any section we have—

$$Z = \frac{I}{y} \quad (462)$$

$$PZ = M = \frac{PI}{y} \quad (463)$$

$$I_p = I + AV^2 \quad (464)$$

And for an I beam, as shown in fig. 902,

$$= \frac{bh^3 + b_1h_1^3}{12} \quad (472)$$

which also applies to the channel section, fig. 903.

Now in large cases of beams, the actual weight of the built-up girder will be in itself a considerable part of the total load, and must be included in our calculations.

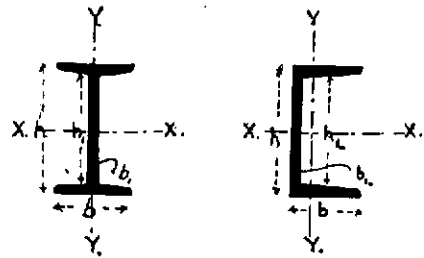


Fig. 902. Fig. 903.

Various formulæ and coefficients have been put forward, but generally, if W = total load on a beam (distributed) if central use, $2W$,

$d = 1/15$ span = proposed depth,

l = span,

S = safe stress in tons per sq. in.,

and C = a coefficient = 1200,

w = weight of beam in tons (or same unit as W and S),

$$= \frac{Wl^2}{C \times S \times d - l^2} \quad (473)$$

Or if we have any girder and know its sectional area in square inches = A , the probable weight of that girder in lbs. per yard

$$= 10A \quad (474)$$

Another way of finding the probable weight of any girder per foot-run

$$= w_1 = 2 \frac{Wl}{d} \quad (475)$$

This applies to an ordinary I beam.

$$\text{For a box girder} \quad w = \frac{Wl}{375} \times \sqrt{\frac{l}{d}} \quad (476)$$

$$\text{and for a plate girder} \quad \frac{Wl}{400} \times \sqrt{\frac{l}{d}} \quad (477)$$

And conversely, if $f = 6\frac{1}{2}$,

$$W = \frac{fad}{1.5l} \quad (478)$$

a referring to area of top flange.

We have already discussed the effect of fixing a beam, that is, it causes principally a higher safe load and three possible points of rupture. In no case whatsoever should any cast-iron beam be fixed. Cast-iron beams have rapidly fallen into disuse, but when designing it is a good idea to make the bottom flange equal in area to the distributed load in tons (square inches), the top flange being anything greater than $\frac{1}{4}$ of this value.

We shall return to the question.

Now in some cases we shall require web stiffeners in a built-up box or plate girder to take up the extra amount of shear which usually comes on

the web towards the supports. By any usual method find the reaction on the supports of the proposed beam = R . Let D = depth of web, then we have a value

$$x = \frac{R}{D} \quad (479)$$

Let y be the permissible shear stress = 5 tons per sq. in.

Then $\frac{x}{y}$ = required thickness of web in inches. If the proposed web fails to comply with this condition, the extra thickness required must be supplied by web stiffeners.

Now if we know the bending moment of a beam at any point, and divide by the effective depth of that beam (viz. depth between centres of flanges), we shall have the stress set up in the flanges at that point. Hence for a distributed load we shall have a gradual diminution of this value towards the ends of the beam in parabolic form. To the beginner this will naturally suggest diminishing the flanges towards the ends for the sake of economy. Practical engineers have found, however, that it is the best practice not to do this, but in place of so doing the depth of the web is decreased instead uniformly towards the supports, which gives us the commonly-called fish-bellied girder used for a variety of purposes, especially overhead travelling cranes which are designed for a maximum centrally applied load.

Our previous remarks on riveting also apply to girders. Of course zigzag riveting is in most cases preferable to chain riveting, because the effective depth will undergo less reduction. In cases such as figs. 868 to 875 special care must be taken over those rivets which are under shear, allowing only 4 tons per sq. in. for those in single shear and $7\frac{1}{2}$ for those in double. The practice of notching, practised by some engineers, tends, however, to take much of the load off these rivets. We also know that towards the end of built-up girders we find a larger amount of rivets, for obvious reasons of shear; but the tendency of reducing this pitch towards the centre must not be carried too far; 16 times the diameter is quite the maximum permissible. This is not because we shall have insufficient strength, but for the reason of buckling occurring, and causing the two or more riveted plates to separate and allow of the admission of air, which will tend to hasten any corrosion. Now it was stated that to find the flange stress in any beam we divided the maximum value of the BM by the depth, which gave us the maximum flange stress. This will very often be too much for a single joist, and yet to use a larger size would not be economy. What do we do then? will be asked. Flange plates will be riveted on to provide for this extra stress in a manner which we are going to describe. It is, however, entirely a matter for the engineer to decide whether it will be cheaper to install the extra heavy joist, or rivet two or more plates on. With small loads the former will no doubt be the case; but when we come to large built-up girders, plates will invariably have to be used. The plates should, however, not have much side overlap, or else we sacrifice stiffness in the beam, which is just as important as mere strength, which is also a reason for the box girder being in most cases of structural work, preferable to the plate girder. We must always be careful to deduct the area of the rivet-holes. As we pointed out in our example, a girder is very often required to carry a wall of a building to allow of an entrance, shop window, or other case. The beam is called a *bressummer*.

Architects usually take the whole wall as solid into consideration when

calculating the total load. This is a practice to be advocated as it errs on the side of safety, but it is not done in contractors' offices, where the all-important consideration is profit. Hence great care is necessary when examining tenders and the usual way to calculate this value is shown in fig. 904, the windows, of course, being deducted. The shaded area alone is calculated.

If no windows occur, upon the span as base, draw an equilateral triangle and calculate upon the enclosed portion.

In the same way as girders and beams, we can calculate for cantilevers and semibeams. Knowing the method of loading, find the maximum bending moment, which usually occurs at the support. Thence we find the flange stress by dividing by the proposed depth, and hence the sectional area.

For this purpose a tee section is usually selected, or two angles back to back, sometimes with a web plate. Owing to the rapid decrease of stress towards the ends of cantilevers, except in very small cases they are rarely

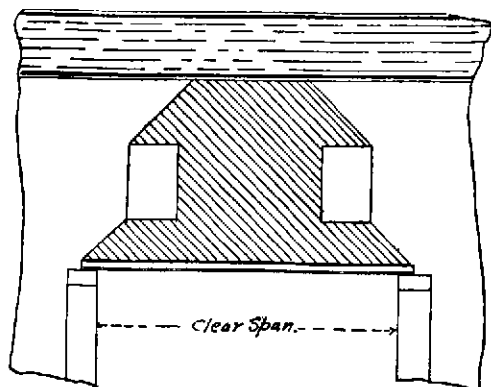


FIG. 904.

made of one section or an uniform depth. When, however, a single joist is used, we can select one from a section book capable of standing four times the proposed load of the cantilever when used as a beam uniformly loaded. We can then easily see what a lavish waste of material we shall have. To reduce this and yet allow of a single member being used, cast-iron brackets are sometimes used, but they cannot be advised. They do not really add much strength.

Now the practice of constructing floors of rolled-steel joists filled in with monolithic concrete still survives. It is a practice about which much good may be said, although advocates of concrete steel construction readily condemn it, and adhere to such principles as have been set forth in Chapter XX. A particular case, however, of economy of such construction is when old steel railway metals are obtainable for the purpose, as they are usually cheap and readily purchased. The rust usually thereon has a decidedly beneficial effect on the mutual adhesion of concrete and steel. A very strong and cheap floor can be made in this way; in fact, many an economical structure can be made of old railway and tram rails. Many examples will present themselves to the intelligent engineer. The writer recently designed a boat-slip which had piles mainly of old steel railway metals (80 lb. flange rails), and tramway rails as stringers. It has withstood the severest gales, and was very cheaply executed. For ease of erection in country places, structures of concrete and old rails are hard to beat. So important, in fact, is this question that we will take the trouble to investigate a special case.

Calculate the weight an old railway metal will safely carry, loaded in the centre for a 10 ft. span.

The conditions of stability of any beam are given by the equation $M = R$, or bending moment = moment of resistance. In the case of a beam carrying

a distributed load, the bending moment $M = \frac{Wl}{8}$, where M = ton-in., W = load in tons, l = span in inches. The moment of resistance is made up of $\frac{I}{y} f$ or ZC , which are different ways of expressing the same thing, I = moment of inertia, y = distance from neutral axis to furthest edge of section, and $\frac{I}{y} = Z$ = section

modulus, $f = C$ = modulus of transverse strength, commonly called extreme fibre stress. The moment of inertia can be obtained as shown on page 99.

Using the figures obtained, $\frac{I}{y} = \frac{28}{3.05} = 9.18 = Z$. Allowing for an extreme fibre

stress of say 7 tons per sq. in., and taking $\frac{Wl}{8} = ZC$, we have $W = \frac{8ZC}{l} =$

$8 \times 9.18 \times 7 \div 10 \times 12 = 4.28$ tons safe load distributed on a single old railway metal, or,

say $8\frac{1}{2}$ tons on a pair bolted together through the webs.

To find the weight of a steel rail in lbs. per yd. we multiply the area by 10. It usually happens, however, that the weight in lbs. per yd. is given; it then remains to find the sectional area by dividing this value by 10.

Returning to the question of joists embedded in concrete, a joist when so treated will be found to be nearly twice as strong as an ordinary one; in fact, any ordinary single joist has its limit of span at twenty times its depth, but if it is encased in concrete up to its full depth this may be increased to 24, to depth and a half 30, and twice the depth 36.

It is usual in such cases to provide for the actual floor loading plus 1 cwt. per sq. ft. area. It is not necessary to place the joist too near to parallel walls, because they will not be subject to much load when so placed. The metal should be always economically distributed.

Let us take any similar floor. We are going to make it on the principle of spanning the opening between the two main girders with steel rails 90 lb. per yd., and are going to provide for a total area of steel = $\frac{1}{10}$ of that of the concrete.

What will be the safe load on the floor = W ?

Let the concrete be t ins. thick.

„ the span be S ft.

„ W = cwt.

$$W = \frac{12t^2}{S^2} \quad (480)$$

which may be easily transposed to give t . The joist would be preferably placed 1 in. from the bottom of the floor slab.

We shall now illustrate to the reader the foregoing principles by means of practical examples.

Let us take a very simple case.

A house is being converted into a shop, and is having 15 ft. of the main wall cut away to make room for a suitable window, the remainder of the walls being supported on a single steel joist. The main wall above the level of the girder is 20 ft. high and 12 in. thick, made of stone. That is to say, we have $20 \times 15 \times 1 = 300$ cubic ft. of masonry to support, supposing no windows to exist in that side of the house. If they did, their weight would be deducted. Now masonry is usually stated to weigh 140 lbs. per ft.-cube, and $140 \times 300 = 42,000$ lbs.

Now this is obviously a distributed load, and any beam subject to a distributed load has a $BM = \frac{Wl}{8}$, where W = total load and l = span in lb.-ft. units.

$$\therefore \frac{42000 \times 15}{8} = 78750 \text{ lb.-ft., which is the max. bending moment.}$$

Say we make the depth of the joist 12 in. = D = ft. Then the stress in the flanges

$$= \frac{WL}{8D} \quad (481)$$

= 78,750 lbs.

Allowing a factor of safety of $4\frac{1}{2}$, the safe load would be 7 tons per sq. in. Therefore the correct sectional area of each flange would be $\frac{78750}{7 \times 2240} = \text{nearly}$

5 sq. in. Therefore we may safely use a 12-in. $\times$ 6-in. $\times$ 54 lbs. rolled-steel joist, having a mean flange thickness of .883 in. = 5.398 sq. in. The web would be $\frac{1}{2}$ in. thick, and the standard section is known as B.S.B. (British Standard Beams) 22. Of course, knowing the distributed load, we could select a section at will from a book of steel joists. The example is given, however, to show how such calculations would be made without a section book.

If we were going to use a cast-iron beam we would have to design it as follows:—

Let D remain 12 in. The stresses set up in the flanges = 78,750 lbs. = say 35 tons; cast-iron in tension can only stand safely $1\frac{1}{2}$ tons per sq. in. Therefore the bottom flange of the beam will have to be 27 sq. in. in sectional area, while the top flange would be 3.5 sq. in., allowing 10 tons per sq. in. compression.

However, it is inadvisable to make the area

of top flange less than $\frac{1}{4}$ the area of the bottom one. So we would make it, say, 6 sq. in. $\frac{1}{4.5}$ section area of bottom flange, being the most economical

section to prevent the upper flange yielding first. The web is made the same thickness as the flange it adjoins. We shall therefore make the beam 12 in. deep, top flange 6 in. $\times$ 1 in., and bottom flange 12 in. $\times$ $2\frac{1}{4}$ in. The section is shown in fig. 905.

The use of cast-iron beams has, however, greatly gone out of vogue, principally owing to the cheapness of rolled-steel joists, and their ease of adaptability to particular cases, and the natural uncertainty of cast iron when under load. The example, however, is interesting.

Let us take the following case:—

An overhead hand-worked travelling crane is being designed for a contractor's yard. The maximum load to be lifted = 8 tons, and the crab, rail, etc., weigh 2 tons = a total load of 10 tons, centrally applied. Owing, however, to the suddenness of application, we shall calculate upon twice this amount = 20 tons. The span is 25 ft., and the arrangement is shown diagrammatically in fig. 906, calculating upon the load being greatest when the crab is in the centre.

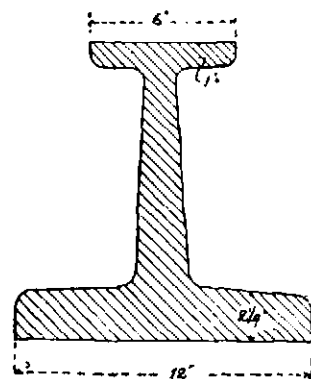
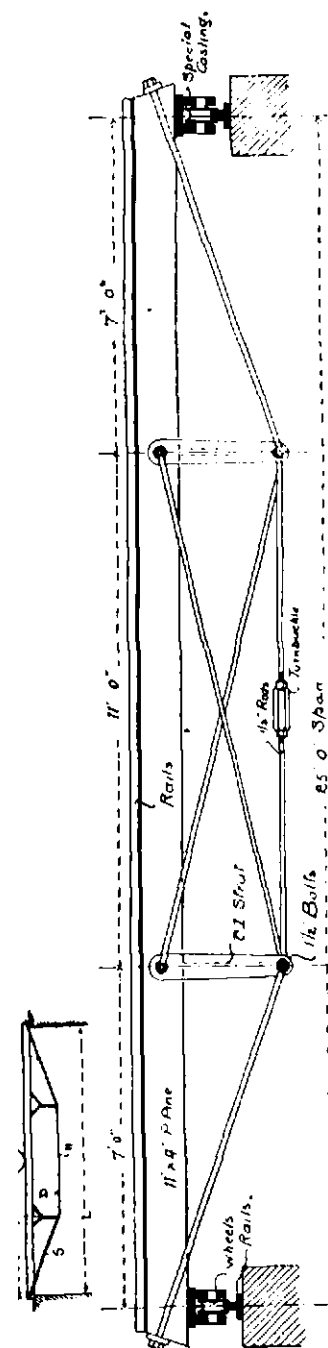


FIG. 905.



FIGS. 906 and 907.

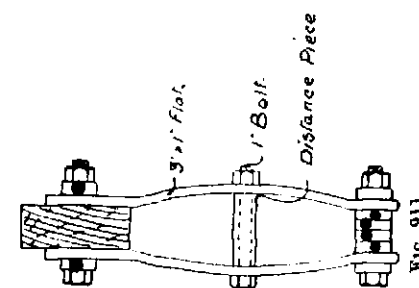


FIG. 911.

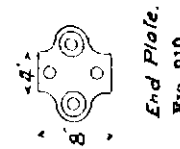


FIG. 910.

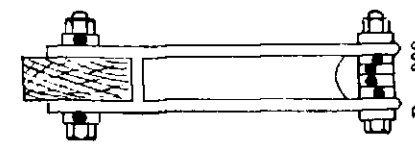


FIG. 908.

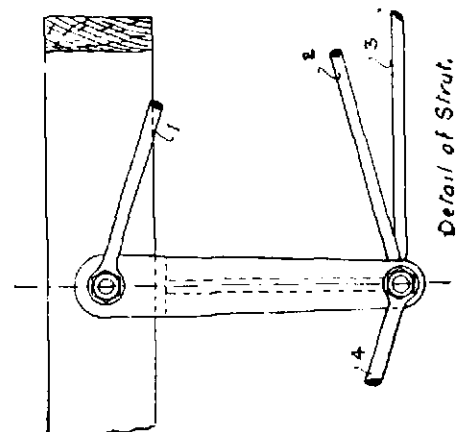


FIG. 908.

Let T = horizontal thrust in the beam in tons due to load.

„ S = strain in inclined tension rods due to load.

„ W = 10 tons on each beam, centrally applied.

It is required to design the beams from these data.

L = span = 25 ft. and D = 3 ft., and l as shown = 7 ft.

$$\text{Then } T = \frac{WL}{4l} \quad (482)$$

$$\text{and } S = \frac{T}{l} \sqrt{l^2 + D^2} \quad (483)$$

$$T = \frac{10 \times 25}{4 \times 3} = \frac{250}{12} = \text{say } 20 \text{ tons,}$$

$$\text{and } S = \frac{20}{7} \sqrt{(7 \times 7) + (3 \times 3)} = 2.9 \sqrt{58} = 2.9 \times 7.61 = \text{say } 22 \text{ tons.}$$

Each strut should be designed to carry the maximum dead load, say 10 tons (*vide* figs. 906 to 911). Now good pitch pine will stand 1000 lbs. per sq. in., and $T = 20$ tons = 44,800 lbs. $\div$ 1000 = say 44 sq. in. Say an 11-in. $\times$ 4-in. joist is used. Again, the stays S will have to stand 22 tons = 11 tons each. Say we use a round steel bar of safe tensile strength of 6 tons per sq. in. = say 3½ sq. in., two 1½ in. diameter rods would be used, and we should probably use the same for the stay H , but we need to screw plus threads on the ends for the turn-buckle to work on. If desired, a rolled tee section could be used in place of the pitch pine, having a total sectional area of say 3½ sq. in., say 4 $\times$ 4 $\times$ ½ in. Cast-iron struts could be replaced by two flats 3 in. $\times$ 1 in., kept apart by distance pieces as shown in fig. 911.

We shall now consider a more complicated case—that of a plate box girder designed to carry a load (dead and uniformly distributed) of 120 tons over a span of 60 ft. Limiting the depth to $\frac{1}{20}$ of the span = 3 ft., while the width of flange is limited to 2 ft., it will be necessary to calculate the approximate weight of this girder, using the Equation

$$W_g = \frac{WL}{400} \times \sqrt{\frac{L}{d}} \quad (484)$$

where W_g = weight of girder in tons,

„ W = distributed load = 120 tons,

L = 60 ft. = span, and d = depth of girder in inches = 36.

$$\frac{120 \times 60}{400} \times \sqrt{\frac{60}{36}} = 18 \times \sqrt{\frac{5}{3}} = 18 \times \sqrt{1.66} = 18 \times 1.30 = \text{say } 24 \text{ tons.}$$

Total load = 144 tons.

We should allow $\frac{1}{10}$ extra span for bearing, making total span = 63 ft.

Now to find the stress in flanges use Equation 481

$$= \frac{WL}{8D} = \frac{144 \times 63}{8 \times 3} = 378 \text{ tons.}$$

Allowing 5 tons per sq. in. on each flange we have

$$\frac{378}{5} = 75\frac{1}{2} \text{ sq. in., say.}$$

But we must also consider shear in such a large girder.

The max. value = reaction at supports = $\frac{144}{2} = 72$ tons.

We will suppose that the depth of web between rivet-holes = 30 in., and as there are two webs, and allowing 3 tons per sq. in. as safe shear stress, the thickness of web plates

$$= \frac{72}{3 \times 26 \times 2} = \frac{6}{13}, \text{ say } \frac{1}{2} \text{ in. plates.}$$

We shall attach the web plates to flanges by 4½ $\times$ 4½ $\times$ ½ angle bars and by rivets in double shear, and using 1-in. rivets which have a sectional area of, say, 0.8 sq. in., and allowing 6½ tons safe stress, 6.5 $\times$ 0.8 = 5.20 tons.

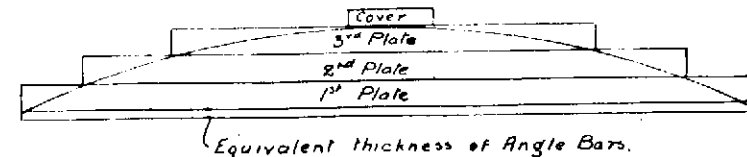
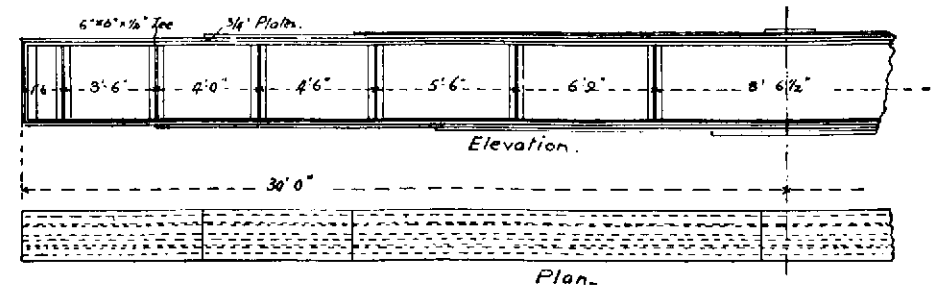


FIG. 912.

$\therefore$ number of rivets per ft.-run = $\frac{72}{2} \div 5.2 = \frac{36}{5.2} = 7$ rivets per ft.-run; that is to

say, the pitch of the rivets will be $\frac{2 \times 12}{7} = \frac{24}{7} = 3\frac{1}{2}$ say (2 being for 2 webs),

and 12 = 12 in. = 1 ft. for rivets per ft.-run, which, for safety at the ends, owing to increased shear, should be reduced $\frac{1}{3}$ = say 2½ in. pitch.



FIGS. 913 AND 914.

Now the width of flange is limited to 2 ft. Therefore at the centre, in order to cope with the max. BM, we have to provide 75½ sq. in. (above)

$= \frac{75\frac{1}{2}}{24} = 3$ in., which would be provided by four ¾-in. plates + 4½ $\times$ 4 $\times$ ½ = area of top side of angles = 8 sq. in., which will more than cover the deficiency of 75½, or, in other words, the equivalent plate thickness of 4 angles = $\frac{4(4\frac{1}{2} \times \frac{1}{2})}{24}$

$$= \frac{9}{24} = \frac{1}{2.66}, \text{ say } \frac{1}{2} \text{ in.}$$

Owing to the decrease of the bending moment towards the supports, the flange plates may also be reduced. At the centre we have four ¾ in. plates. Draw fig. 912 to a distorted scale as shown, and by any usual method draw a

Dividing this by max. depth = 6 ft. gives us 42 tons stress on flanges at centre.

Now bending moment at any point, say 5 ft. from centre, $= W\left(\frac{L}{2} - x\right)$,

where $x = 5$ ft. $= 10\left(\frac{60}{2} - 5\right) = 10 \times 25 = 250 \div 42 = 5$ ft. 11 in.

Again, $10 \times 20 = 200 \div 42 = 4$ ft. 10 in.

and in the same way we obtain three more values of the depth

$= 3$ ft. 8 in., 2 ft. 6 in., and 1 ft. 3 in.

Allowing a stress of 6 tons per sq. in. in each flange, we want $\frac{42}{6} = 7$ sq. in. of metal.

This would be best provided for by two 4-in. $\times$ 4-in. $\times$ $\frac{3}{8}$ -in. angles $= 2 \times 2.87$

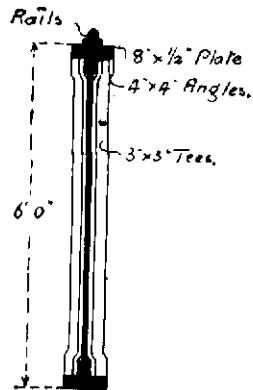


Fig. 917.

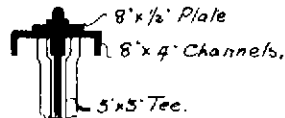


Fig. 918.

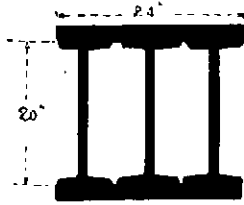


Fig. 919.

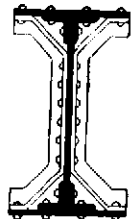


Fig. 920.

$= 5.74$ sq. in., and a cover-plate of $\frac{1}{2}$ -in. metal, $8\frac{1}{2}$ in. wide $= 4.25$ sq. in. + 5.74 , will give ample area.

Using 1-in. rivets, we find the minimum pitch of them to be (using Equation 488)

$$= P = \frac{f d l}{S} = \frac{3.5 \times 12}{10} = 4.2, \text{ say } 4 \text{ in. at the ends,}$$

which might be reduced at the centre to $\frac{3.5 \times 70}{10} = 21$ in., but in practice the

maximum pitch would be about 8 in. Owing, however, to lack of lateral stiffness in this type of girder, it would be advisable, instead of the 4-in. $\times$ 4-in. angles stated, to use 8-in. $\times$ 4-in. channels, as shown by fig. 918, instead of fig. 917, unless the two girders were cross-braced. Web stiffeners of 3-in. $\times$ 3-in. tees would be placed as shown, and 3 $\times$ 3 angles at the very ends, to which $\frac{3}{8}$ -in. cover-plates could be bolted.

The elevation of the girder satisfying these conditions is shown in fig. 916. Light diagonal bracing could be employed with advantage, as shown in the figure.

We will now work out an example of a lattice girder, a process which would present more elaborate calculations; at the same time they do not present any great mathematical difficulties.

We will take for our example a lattice girder for a span of 150 ft., which is to become one of the main girders of a road bridge. The loads having been duly calculated by usual methods, it has been found that the equivalent dead load on each girder is 150 tons, and that the load on each joint $= 15 = 150 \div 10$ total number of divisions in the span. The first thing to do is to sketch a skeleton diagram of the proposed girder as in fig. 922, placing numbers thereon as shown, the actual girder appearing in fig. 923.

Let $N = 10$.

„ $l = \text{span} = 150$ ft.

„ $K = \text{depth} = \frac{l}{10} = 15$ ft.

„ $H_n = \text{thrust or tension in booms.}$

„ $V_n = \text{stresses in the vertical rods.}$

„ $T_n = \text{stresses in diagonals.}$

the suffix n referring to the number of the joint in question.

First of all let us calculate the value of $S = \text{length of diagonals, } w = \text{the equivalent uniform load on each joint,}$

$$= \sqrt{K^2 + \left(\frac{l}{N}\right)^2} \quad (489)$$

$$= \sqrt{15^2 + \frac{150^2}{10^2}} = \sqrt{15^2 + 15^2} = 15^2 = \sqrt{225 + 225} = \sqrt{450} = 21.21 \text{ ft.}$$

Now $H_5 = \text{max. value of } H,$

$$= \frac{wl}{K} \times \frac{n(n-N)}{2N} \quad (490)$$

$$= \frac{15 \times 150}{15} \times \frac{5(10-5)}{2 \times 10} = \frac{150}{1} \times \frac{25}{28} = 25 \times 7.5 = 188 \text{ tons.}$$

Allowing a stress of 6 tons per sq. in. we have $\frac{188}{6} = 31.6$, for which we shall use two 12-in. $\times$ $3\frac{1}{2}$ -in. channels, having a total sectional area $= 2 \times 9.67 = 19.34$ and a 26-in. $\times$ $\frac{1}{2}$ -in. plate $= 19.34 + 13 = 32.34$ sq. in.

The value of H_n will be somewhat reduced towards the ends, but it will be economical to keep this section right through. The continuous plate may, however, be dispensed with as the stresses get reduced, and short lengths of cover-plate substituted if great economy is aimed at.

We now proceed with the diagonals.

Diagonal at 1.

$$T_1 = \frac{WS}{2K} \left(\frac{N+1}{2} - n \right) + \frac{WS}{2K} \left(\frac{(N-n)(N-n+1)}{2N} \right) \quad (491)$$

$$= \frac{15 \times 21.21}{2 \times 15} \left(\frac{10+1}{2} - 1 \right) + \frac{15 \times 21.21}{2 \times 15} \left(\frac{(10-1)(10-1+1)}{2 \times 10} \right)$$

$$= (10.60 \times 4.5) + (10.60 \times 4.5)$$

$$= 47.7 + 47.7 = 95.4 \div 6 = \text{say } 16 \text{ sq. in.,}$$

parabolic curve of height = 4 plates and span = effective span of girder. Where it cuts the plates will give the proper length to which they must be cut. Steel plates can usually be readily obtained up to 30 cwt. in weight. This will usually necessitate some butt joints in the flange and web plates. Those in the top flange do not need much attention owing to the stress being compressive; 6 rivets in each row each side would be ample; but owing to the compressive stress in the bottom flange must be calculated as follows:—

The plate is $\frac{3}{4}$ in. and 24 in. wide = 18 sq. in. cut through. Each 1-in. rivet has sectional area = .8 sq. in. and $\frac{18}{.8} = 22.5 =$ say 23 rivets, but being an

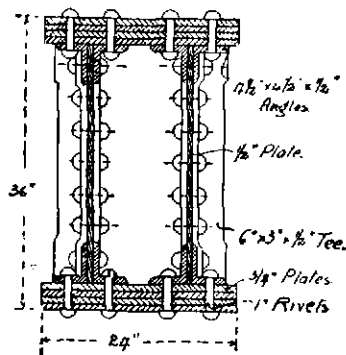


FIG. 915.

inconvenient number, call it 24, and each row has 4 rivets, and there are two ends to each cover. Therefore $\frac{24}{4} \times 4$ (inches pitch) $\times 2 = 48$ in. = 4 ft.

The bottom cover at centre should be 4 ft. long. The cover for web plates would be $\frac{1}{2}$ -in. plates 6 in. wide, having $\frac{3}{4}$ -in. rivets at 4-in. pitch.

Now to take the shear, there will be tee stiffeners between the angles, closed at the ends than centre, 6 in. $\times$ 3 in. $\times$ $\frac{1}{2}$ in. Tees would do well, but at the extreme ends, to which $\frac{1}{2}$ -in. end plates would be bolted by $\frac{3}{4}$ -in. bolts, $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ angles would be used.

The completed design of beam would be as in figs. 913, 914, and 915. It would be interesting to find the deflection of this beam under load.

S = safe stress = 5 tons per sq. in.

L = span in inches = 60×12 .

D = mean depth = 36 in.

E = modulus of elasticity = 13,000 tons.

Then deflection in inches

$$= \frac{SL^2}{4ED} \quad (485)$$

$$= \frac{5 \times 60 \times 12 \times 60 \times 12}{4 \times 13000 \times 36} = 1.385 \text{ in.}$$

The advantage of the box girder is its great lateral stiffness, especially over large spans. The above method of design will be readily applied by the intelligent reader to any form of built-up box or plate girder. The example given is only typical; there may be three webs or only one. This, again, may be built up of plates and angles as in this particular case, or it may be simply rolled joists. Diagrams of two other examples are given in figs. 919 and 920. Other forms will readily present themselves (*vide* fig. 919).

It will be observed from what we have said that the bending moment depends on the load, method of loading and supporting, and the span. The stress varying inversely as the depth and the BM in any section $\div$ depth will give the stress in the flanges at that point. These considerations give rise to that form of design known as the fish-bellied girder, as stated by pro-

portioning the depth to the BM, making the flange stress practically constant along the whole girder. For instance, it is proposed to design the girders for an overhead 10-ton electric travelling crane over a span of 60 ft. The wheel base of the crane is 10 ft.

In this case we may have two cases of actual BM, viz. with load at centre

$$= \frac{WL}{4} \quad (486)$$

and when one-quarter of the wheel base is on either side of the centre

$$= \frac{W}{2L} \left(L - \frac{b}{2} \right)^2 \quad (487)$$

b being length of wheel base, a fact which will depend on ratio of span to wheel base. We can transpose equation to $\frac{W}{2L} \times \frac{L^2}{2}$ for the sake of comparison, in which it will be obvious that as long as $\left(\frac{L-b}{2} \right)^2 - \frac{L^2}{2}$ is equal to or greater than 0, that is, so long as $L \geq 1.866b$, this Formula (No. 487) will give the

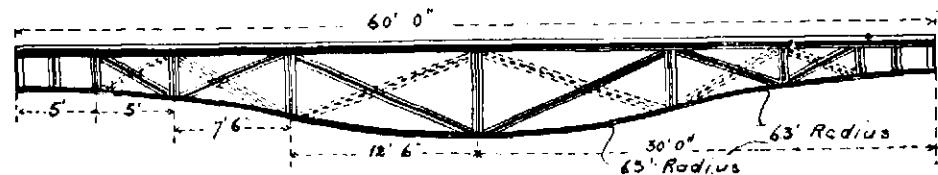


FIG. 916.

greater result. It will be economical to make the max. depth of girder = $\frac{1}{10}$ span.

When designing the flange for such a girder as this, we must also provide for its resisting a lateral force = $\frac{1}{8}$ of the total moving load in addition to the vertical loading (which sets up a lateral force), which is usually resisted by bracing the flanges of the two crane girders together. This lateral thrust is especially noticeable when starting and stopping the crane.

Having decided upon the depth at centre = $\frac{1}{10}$ span = 6 ft., we have to select the other depths and web thicknesses so as to allow the rivets to be driven at a normal pitch.

Then if f = safe shear stress on each rivet in tons,

d = depth rivet centres in ins.

S = shear at end in tons

P = pitch of rivets in ins.

$$\text{then} \quad P = \frac{fd^2}{S} \quad (488)$$

and for 1-in. rivets $f = 4.5 \times .8 = 3.60$, say $3\frac{1}{2}$.

Now let us get the max. flange stress at centre BM (max.)

$$= \frac{W}{2L} \left(L - \frac{b}{2} \right)^2 \quad (\text{Equation 487}).$$

$$\therefore \frac{10}{2 \times 60} \left(60 - \frac{10}{2} \right)^2 = \frac{1}{12} \times 55^2 = \frac{1}{12} \times 3025 = 252 \text{ ton-ft.}$$

which would be provided by

$$\text{four angles } 4 \times 4 \times \frac{5}{8} = 4 \times 4.61 = 18.44 \text{ sq. in.}$$

The angles would be laced.

Proceeding in the same way with diagonal at

$$2 = T_2 = (10.60 \times 3.5) + (10.60 \times 3.6) = 37.1 + 38.16 = 75.26 \div 6 = 12.54,$$

which will be provided for by

$$\text{four angles } 4 \times 3 \times \frac{1}{2} = 4 \times 3.251 = 13.004 \text{ sq. in.}$$

Again, at point 3, we treat the diagonal likewise = T_3

$$= (10.60 \times 2.5) + (10.60 \times 2.8) = 26.5 + 29.68 = 56.18 \div 6 = 9.36,$$

and we shall use

$$\text{four angles } 4 \times 3 \times \frac{3}{8} = 2.485 \times 4 = 9.940.$$

Finally, the diagonals terminating at the centre, we have

$$T_4 = (10.60 \times 1.5) + (10.60 \times 2.1) = 15.9 + 22.26 = 38.16 \div 6 = 6.36,$$

for which it would be wise to use the same section as before, namely,

$$\text{four angles } 4\text{-in.} \times 3\text{-in.} \times \frac{3}{8}\text{-in.}$$

Again, on the end post we have a stress

$$= W \frac{aS}{KL} = T_0$$

where a = distance of centre of post from the centre of girder = 70 ft., $S = 21.21$ (as already found by Equation 489).

$K = 15$ ft. depth of girder,

$L = \text{span} = 150$ ft.

$$\therefore T_0 = 150 \frac{21.21 \times 70}{15 \times 150} = 98 \div 6 = \text{nearly } 17 \text{ sq. in.}$$

We shall use the same section as for booms, viz. two 12-in. $\times$ 3½-in. channels but the 26-in. $\times$ ½-in. plate is not required, so they would be laced with light latticework, or the plate could be retained, if preferred, to save labour.

We now consider the vertical members.

First let us take the central bar,

$$V = w \left(\frac{N}{2} + 1 \right) = 15 \times \left(\frac{10}{2} + 1 \right) = 15 \times 6 = 90 \text{ tons.}$$

Say we try a 16-in. $\times$ 6-in. I beam. Using Rankine's formula, and considering it as fixed at both ends, we find the safe load is equal to

$$P = \frac{fa}{1 + C \left(\frac{l^2}{r^2} \right)},$$

a formula now so familiar with the reader as to need no further explanation,

$$C = \frac{1}{60000}.$$

$$\therefore P = \frac{6 \times 18.23}{1 + \frac{1}{60000} \times \frac{32400}{4}} = \frac{109.38}{1.15} = 95 \text{ tons.}$$

Therefore our section will do, owing to 6 tons being a low safe stress. Proceeding with the remaining diagonals, we have to use certain coefficients.

These coefficients are as follows:—

For diagonal at 1 = 10.

" " " 2 = 28.

" " " 3 = 21.

" " " 4 = 15.

To get the stresses we find the load on each joint = as before 15 tons; $\times$ by the coefficient, and divide by the number of bays in the girder. Therefore we have V_1 = stress on diagonal at 1 = $15 \times 10 = 150 \div 10 = 15$ tons.

Using Rankine's formula as above, we find that four 3½-in. $\times$ 3-in. angles will satisfy our requirements.

At 2 $V_2 = 28 \times 15 = 420 \div 10 = 42$ tons.

Likewise $V_3 = 21 \times 15 = 315 \div 10 = 31.5$ tons.

Also $V_4 = 15 \times 15 = 225 \div 10 = 22.5$ tons.

Repeating previous calculations, we find that for verticals in a girder of this description we should get the most economical results by using the 16-in. I beam right through, except for the end ones, because the saving of labour would more than compensate for the extra weight of material used.

It may here be remarked that, for the verticals, the use of certain predetermined coefficients saves much labour and gives very accurate results. For the other members, however, these do not give such reliable results as the methods set forth, and are not here given.

For sake of comparison we can work out the values for V_2 , V_3 , and V_4 as follows, getting a slight difference in stress of about 2 tons, which would only mean a saving of metal of about ½ sq. in. one way or the other, according to which gave the smallest result.

$$\text{Thus } V_4 = \frac{w(N+1)}{2} - \frac{w(N-n)(N-n+1)}{2N} \quad (492)$$

$$= 7.5(5.5 - 4) + (7.5 \times 1.5) = 11.25 + 11.25$$

22.5 = the same result as before.

$$\text{Again, } V_3 = 7.5 \left(\frac{11}{2} - 3 \right) + 7.5 \left(\frac{(10-3)(10-4)}{2 \times 10} \right)$$

$$= (7.5 \times 2.5) + (7.5 \times 2.1) = 18.75 + 15.75$$

$$= 34.50 \text{ tons} = 3 \text{ tons difference,}$$

$$\text{and } V_2 = 7.5 \left(\frac{11}{2} - 2 \right) + 7.5 \times \left(\frac{(10-2)(10-3)}{2 \times 10} \right)$$

$$= (7.5 \times 3.5) + (7.5 \times 2.8) = 26.25 + 21.0$$

$$= 47.25 = 5.25 \text{ tons difference.}$$

Again, if we take the modulus of elasticity of the steel used at 12,000 tons, and the safe stress $= f = 6$ tons, the deflection of our girder under load will be in inches

$$= d = \frac{3L^2f}{EK} \quad (493)$$

where L = span in ft. and K = ft. also,

$$= \frac{3 \times 150 \times 150 \times 6}{12000 \times 15} = \frac{9}{4} = 2\frac{1}{4} \text{ in.}$$

$$= \frac{2.25}{150 \times 12} = \frac{17}{150} = \text{nearly } \frac{1}{800} \text{ of the span.}$$

Where joints in the booms would occur, the plates and riveting would be calculated in a similar way to the example of the box girder, H being the tension in the lower boom and the compression in the upper one.

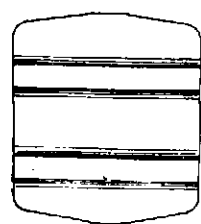


FIG. 921.

The drawings of such a girder designed as above are given in fig. 923. Of course the actual design of the bridge, such as the flooring, wind bracing, etc., is another part of the process, but it would be almost foreign to the article which treats on girders only: 1-in. rivets at 4-in. pitch would probably be used throughout.

We will now conclude this chapter with a few notes on the practical design of girders in general. The span should not greatly exceed twenty-two times the depth, or else excessive stresses will be set up in the girder, necessitating

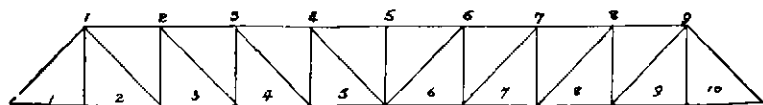


FIG. 922.

reduction of the safe working stress, and probable waste of material due to uneconomical distribution.

Nor should the length greatly exceed twenty times the width of flange, which would cause side yielding. This, however, may be provided for by some form of side bracing. Where parallel joists form the members of a girder, they should be suitably bolted to distance-pieces. These may be special castings, as in fig. 921, or pieces of gun-barrel in light work.

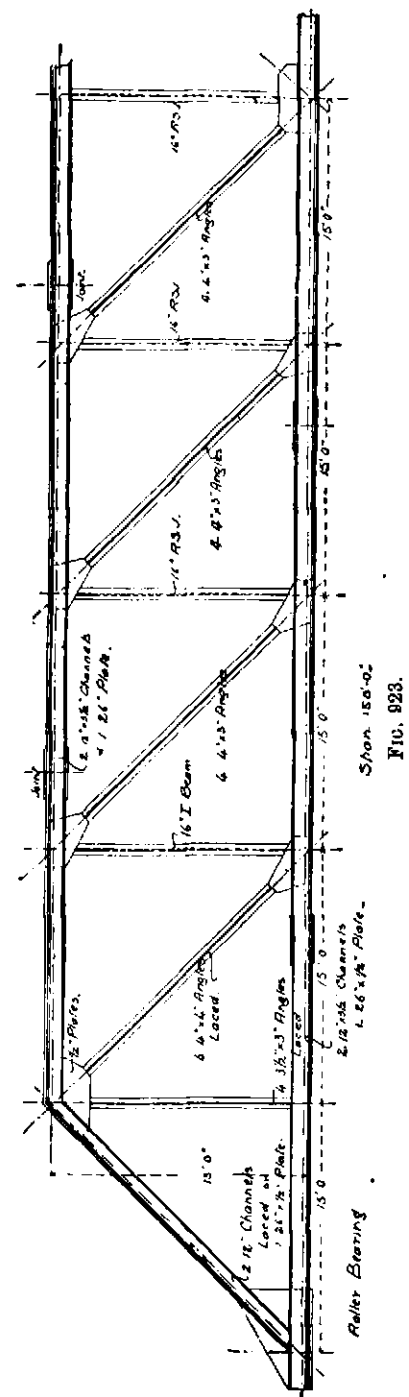
In large box and plate girders stiffeners are required, especially near the supports, due to the increased shear; they are specially necessary when the web plates are less than $\frac{1}{8}$ of the depth, to prevent their buckling. The length of bearing should not be less than the depth of the joist. Pins, when used, should not be less in diameter than $\frac{1}{4}$ the full width of the bar they are in.

Now the following case of a beam may be described as a special one, and is of interest because in the next chapter we are going to show ways and means of carrying pipes in a similar way; but instead of allowing the actual tube to be stressed, we are going to provide for special

steelwork to do this duty. We are assuming the pipe to be full of water. The general arrangement is shown in fig. 924. The total load will then be, say, 12.66 tons; and taking the pipe as a continuous beam, the distribution of the loading will be as marked on the frame diagram (fig. 925), for which fig. 926 is the stress diagram. The bending moments on the pipe will be as shown in fig. 927. In calculating the stresses, only two-thirds the area of the steel may be taken into account in order to allow for loss of strength by riveting. Then the maximum stresses will be $\frac{W}{A} \pm \frac{M}{Z} = \frac{22.5}{12.67} \pm \frac{8.8 \times 12}{110} = 1.77 \pm 0.96 = 2.73$ tons per sq. in. compression maximum, and 0.81 ton per sq. in. compression minimum, in accordance chiefly with the variations of the bending moment shown in fig. 927, and the pipe will therefore be quite safe. Allowing 6.5 tons per sq. in. in tension, the rods will require an area of $\frac{23.0}{6.5} = 3.54$ sq. in., requiring,

say, two $1\frac{1}{4}$ -in. square rods, with large end welded on. The struts must be curved where the pipe bears on them, in order to give full support.

We will return to the important subject of lattice girders. Since writing the earlier pages of this chapter the writer has had the following case of a lattice girder placed before him, which is loaded as shown in fig. 928. The girder, of quite a light type, of similar lines to fig. 930, is for supporting a roof. Required the several stresses thereon. Now to work the stresses in a small girder of this type out completely would be quite unnecessary and take a very long time, because the loading is not symmetrical, and there are so many members to be taken into account. Approximately, the maximum stresses in the flange may be obtained by first finding the maximum bending moment, as shown in figs. 928 and 929, and dividing by the depth of the girder,



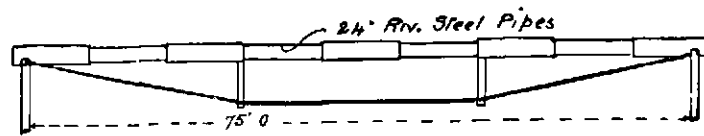


FIG. 924.

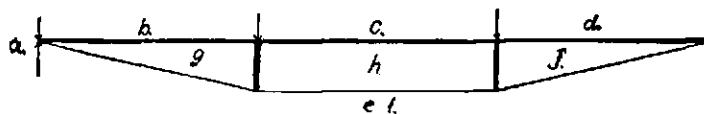


FIG. 925.

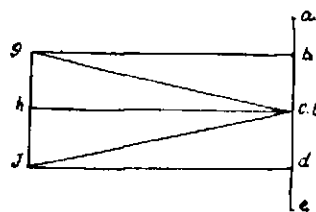


FIG. 926.



FIG. 927.

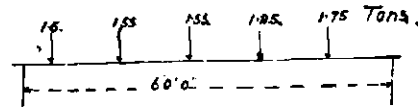


FIG. 928.

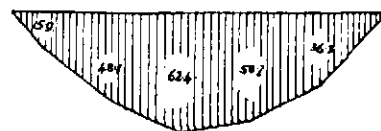
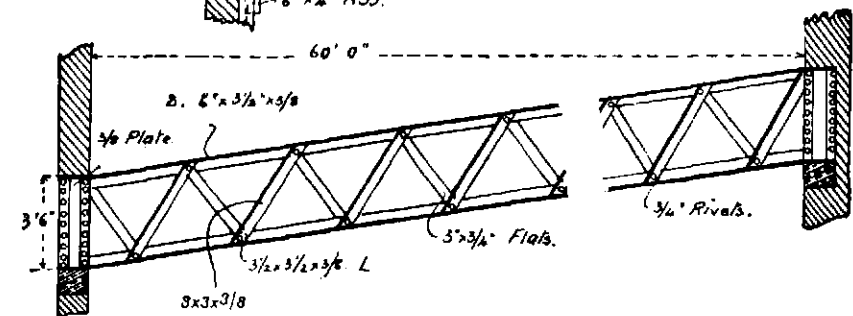
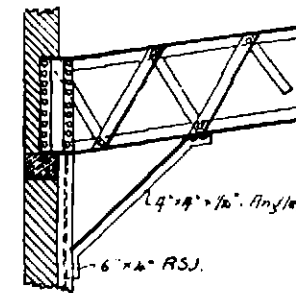


FIG. 929.

as follows: Maximum bending moment = 62.4 ton-ft., depth of girder = 3 ft. 6 in., then stress in flanges = $\frac{62.4}{3.5} = 17.8$ tons. Allowing an ultimate stress of 26 tons per sq. in., the two 6-in. by $3\frac{1}{2}$ -in. by $\frac{3}{8}$ -in. angles will carry $W = \frac{fA}{1 + \frac{1}{a}(\frac{l}{d})^2} = \frac{26 \times 6.46}{1 + \frac{1}{750}(\frac{138}{7})^2} = 110$ tons ultimate load, giving a factor

of safety of $\frac{110}{17.8} = 6.18$. The stress in the 3-in. by 3-in. by $\frac{3}{8}$ -in. end diagonals will be approximately 2.8 tons. The compression member with an un-



FIGS. 930 and 931.

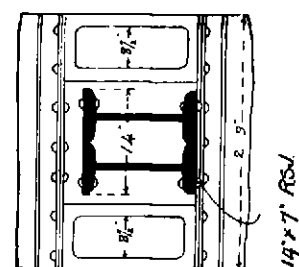
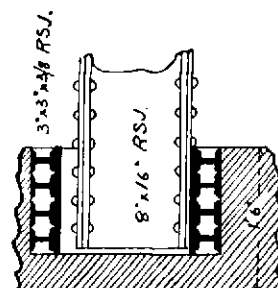
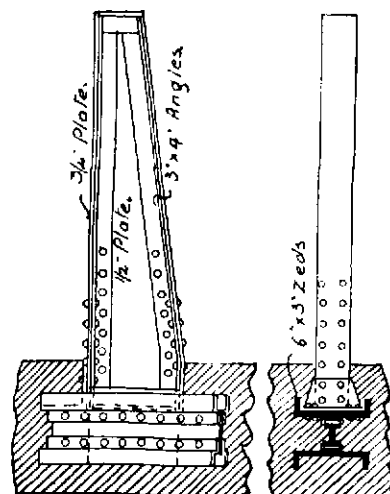
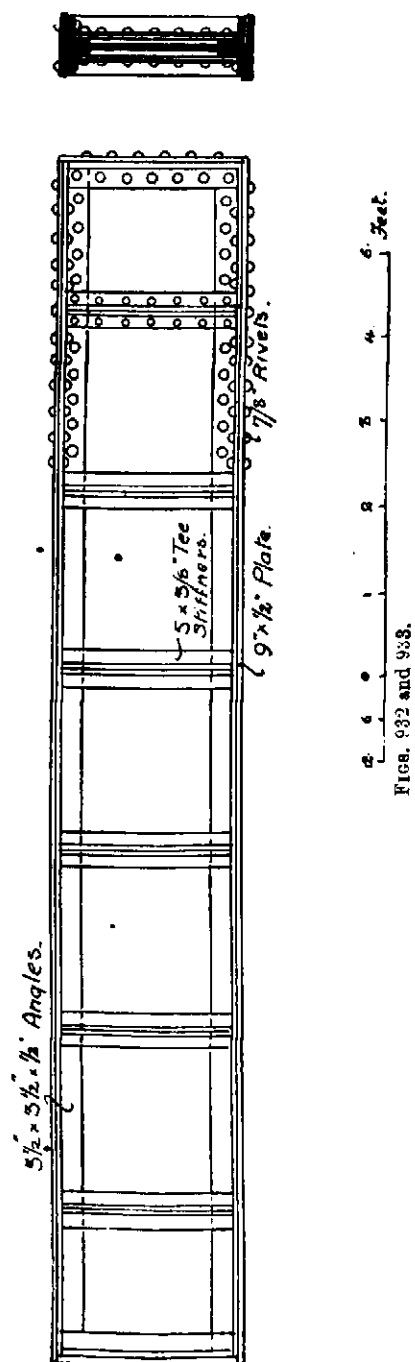
supported length of 9 ft. will take $W = \frac{fA}{1 + \frac{1}{a}(\frac{l}{d})^2} = \frac{26 \times 2.11}{1 + \frac{1}{750}(\frac{108}{2.2})^2} = 12.76$ tons,

giving a factor of safety of $\frac{12.76}{2.8} = 4.55$. The tension member has an area of 1.81 sq. in. after deducting for one rivet-hole, and the area required = $\frac{2.8}{26} =$

0.11 sq. in., giving a factor of safety of $\frac{1.81}{0.11} = 16.45$. These factors seem very high, but no allowance has been made in the loads for wind pressure on the surface of the roof. When this is taken into account the factor of safety will be very small for the top flange of the girder.

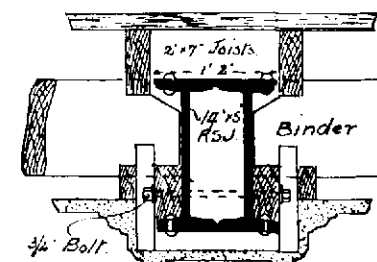
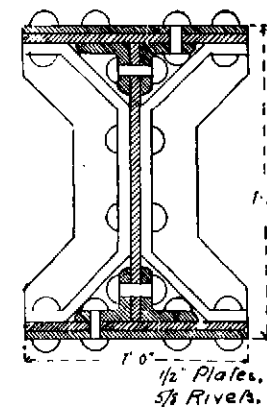
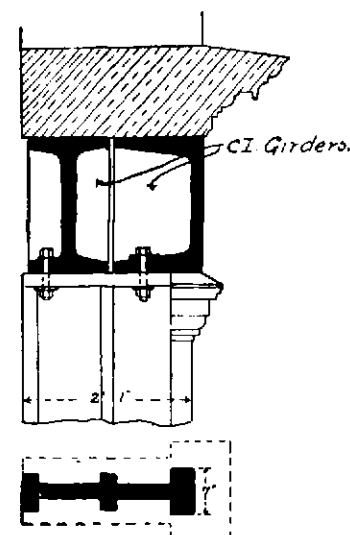
As a further example of girder design, figs. 932 and 933 are given.

Figs. 934, 935, and 936 show a built-up cantilever anchored to a wall. Such an anchorage requires careful design so as to resist toppling over under



imum possible load. Figs. 937 and 938 illustrate a method of providing bracing for a large girder in a wall. Figs. 939 and 940 show a cast-iron column, 941 an example of a built-up girder with cast-iron stiffeners, and 942 a steel binder used in ordinary floor construction.

BEAMS AND GIRDERS.



CHAPTER XXXVIII.

COMBINED STRUCTURES IN IRON AND STEEL.

WE now come to this last category of constructional steelwork. In this chapter we shall give examples of combined structures whose essential constituent is steel, and investigate some methods whereby pipes for sewers and water mains can be taken across rivers. Now in cases like this it is a matter of no little judgment to decide upon the correct place in a book of this kind for these special and so important problems. Mention was made under Sewers of a way of effecting this, but we decided, after much deliberation, that the actual calculations and design are better placed in this division.

We hope, therefore, that this may serve as an apology for a little want of uniformity in some cases in the book.

Laying Pipes across Rivers and Streams.—The means of carrying sewer pipes and water mains across small rivers and ravines is a problem which very often confronts engineers, especially those in municipal practice. It is intended in this chapter to point out to the reader methods of effecting this in a simple, practical, efficient, and at the same time inexpensive way. We are not concerned with work of large magnitude, such as siphon pipes under canals and navigable

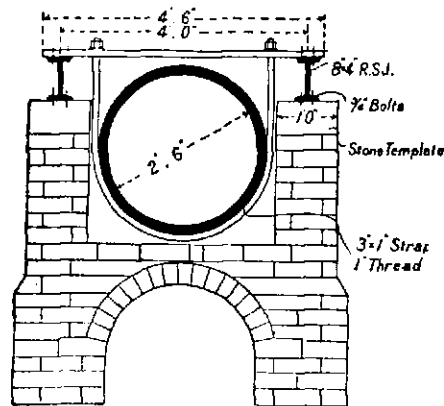


FIG. 943.

streams, nor is it intended to treat upon large aqueducts. Let us take the following case:—

It is required to lay a 30-in. cast-iron flange water main over a shallow stream 60 ft. wide, at a height, say, 4 ft. above maximum flood-level. Referring to figs. 943 and 944, we are placing two piers (4 in all) at 20-ft. centres. They would be made of masonry, brickwork, or concrete. Looking up a table of cast-iron pipes, we find that a 30-in. water main, tested to 300 lbs. pressure, weighs 3.29 cwt. per foot-run, and $3.29 \text{ cwt.} \times 20 = 3.29$ tons (say 3.30 tons) between each span. Again, the sectional area of a 30-in. pipe = 4.91 sq. ft., and $\times 20 = 98.2$ cub. ft. capacity. Now taking water at 62.5 lbs. per cubic foot we have $98.2 \times 62.5 = 6137.5$ lbs., or say 2.8

tons. Therefore $3.30 + 2.80 = 6.10$ tons total load. Referring to any table of properties of rolled steel joists, we find that to support $\frac{6.1}{2}$ tons (= 3.05 tons) if we use two 8-in. $\times$ 4-in. I beams, we shall have a supporting capacity of 7 tons, or nearly 1 ton in excess. These, moreover, would be the most economical sections to use. The weight of these joists per foot-run is 18 lbs. $\therefore$ the total weight of the superstructure is $(6.10 \times 3) + \frac{(18 \times 120)}{2240}$ tons =

19.26 tons, or, allowing for contingencies, straps, etc., say 20 tons. This is distributed over 4 piers, the two middle ones of which will each bear about $6\frac{1}{2}$ tons and the two outer ones $3\frac{1}{2}$. This will fix the area of the piers and foundations. Figs. 943 and 944 show the construction fully detailed and dimensioned.

Let us now take another case, that of the 9-in. water main (spigot-and-socket) laid across a river, the span from centre to centre of supports being

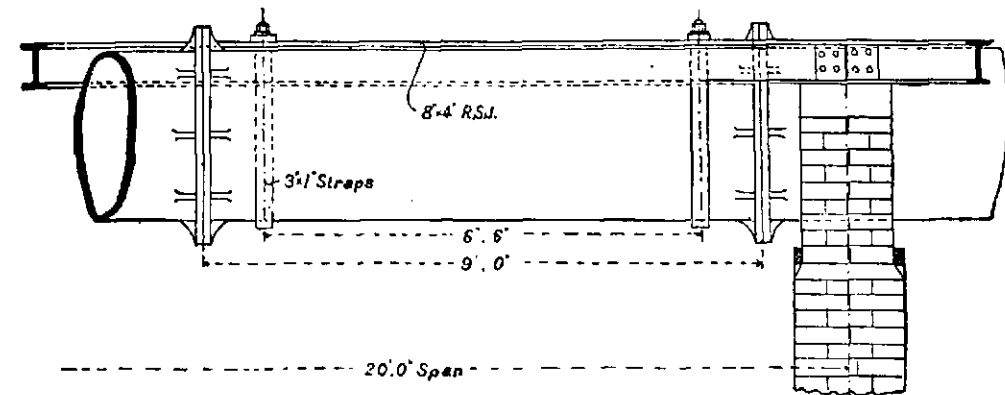


FIG. 944.

50 ft. Now 50 ft. of 9-in. water main, including sockets, weighs (if of $\frac{1}{2}$ -in. metal) 2610 lbs., assuming it to be made up of 9-ft. lengths at 47 lbs. per foot-run, each socket being equal to 1 ft. of pipe. The water in the pipes will weigh 1352 lbs. by the rule of a gallon of water being equal to 10 lbs., and the number of gallons per yard in any pipe being equal to $1-10d^2$ (d = diameter in inches). Therefore total weight = $2610 + 1352 = 3962$ lbs. Now a good economical way of doing this would be to lay a rolled steel joist of suitable section with flanges vertical, so forming a trough in which the pipes would lie. Looking up a table of joists again we find that a 12-in. $\times$ 6-in. $\times$ 54-lbs. I beam would be somewhere near the mark. Say we test the assumption by the familiar equation $\frac{W}{A} \pm \frac{M}{Z}$ so as to see that the compression and tension are within safe limits. Area of joists = 15.88 sq. in. Maximum compression = 7.61 tons. $\therefore \frac{7.61}{15.88} \pm \frac{13.85}{9.4} = .48 + 1.47 = 1.95$ tons per sq. in. compression and 1 ton tension, both easily within safe limits. Figs. 945 to 948 show full details. We are assuming that the supports are to be steel joists as shown. It would naturally be convenient to use

12-in. x 6-in. joist for this if we could. We proceed to test this by the Rankine Gordon formula, as follows:—

$$P = \frac{f}{1 + \frac{1}{AC} \left(\frac{l}{r} \right)^2}$$

f = allowable stress = 10000 lbs.
 $l = 20$. $r = 1.80$. $A = 2$. $C = 9000$.

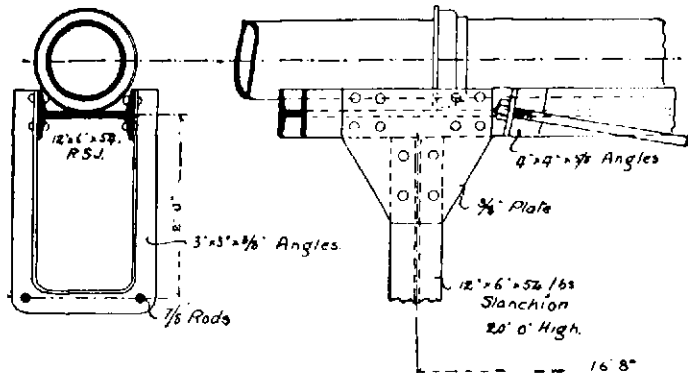


FIG. 945.

FIG. 946.

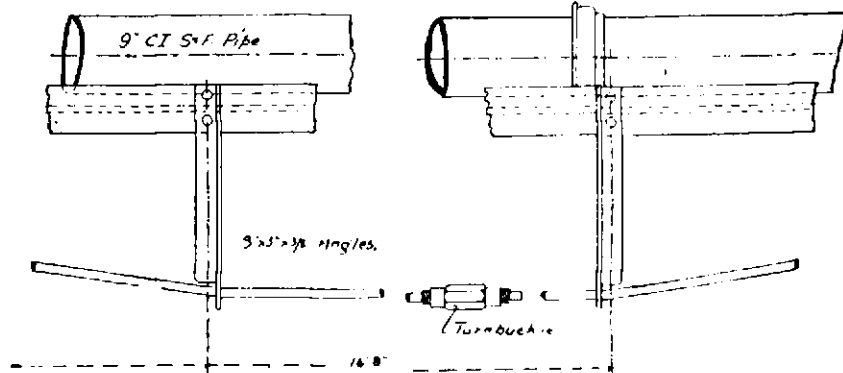


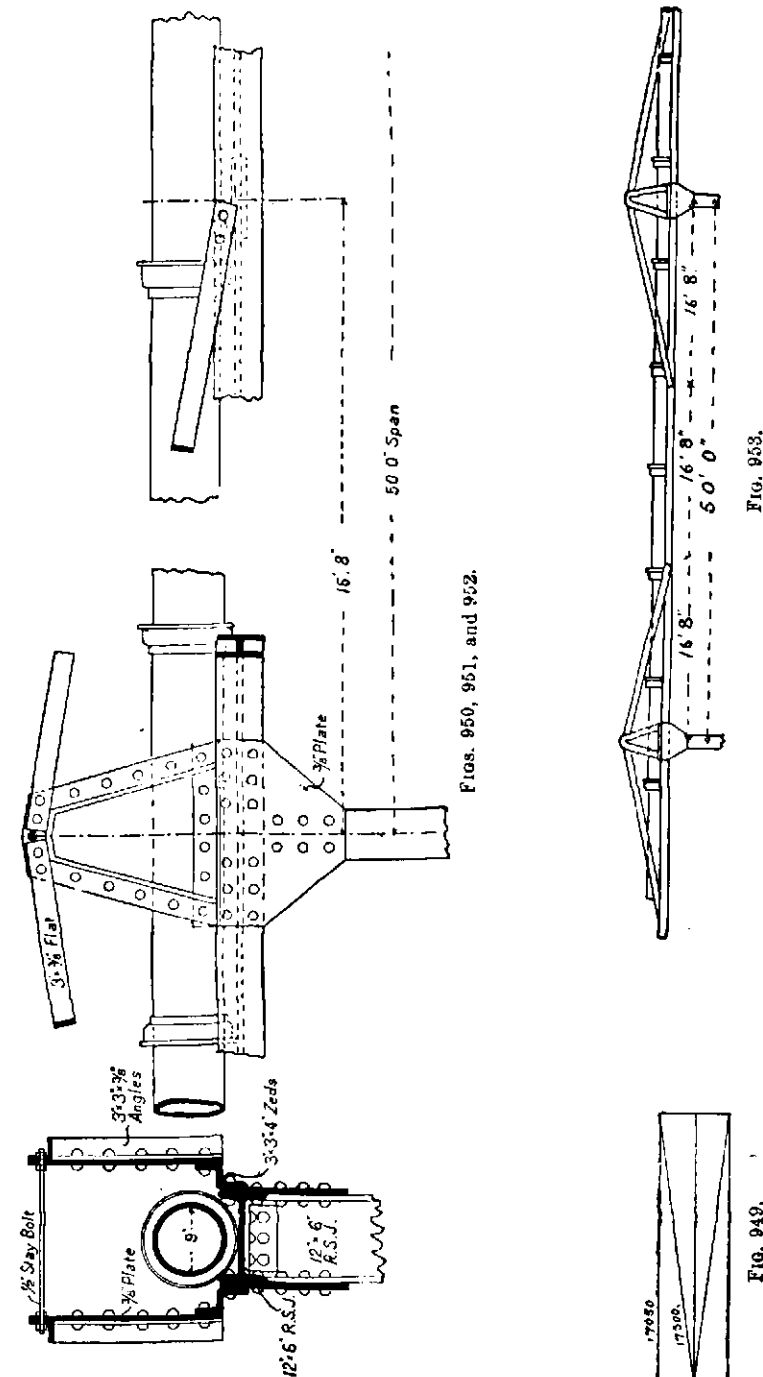
FIG. 947.

FIG. 948.

Then $P = C$ compression per sq. in. =

$$\frac{1000}{1 + \frac{1}{2 \times 9000} \left(\frac{20}{1.8} \right)^2} = 2 \text{ tons.}$$

We may therefore safely use this section. The tension rods may be easiest calculated by the diagram shown in fig. 949, in which we have a tension of 17,300 lbs. = 7.85 tons. Allowing a tension of $6\frac{1}{2}$ tons per sq. in., we want $\frac{7.85}{6.5} = \text{say } 1.25 \text{ sq. in.}$ We should use two $\frac{7}{8}$ -in. rods, which



FIGS. 950, 951, and 952.

FIG. 953.

FIG. 949.

should, however, have "plus" threads, which will necessitate their being thickened out to 1 in. diameter at the ends. The use of the tension rods, involving, as they do, however, a certain amount of smithwork, always unsatisfactory to structural engineering, may be obviated by the construction shown in figs. 950 to 953, by constructing saddles of steel sections and plates, as shown in the drawings. In this case we are using flat bars, the tension in such being slightly more than if they were placed as in fig. 946, viz. about 18,500 lbs., which would easily be borne by two 3-in. $\times$ $\frac{3}{8}$ -in. bars, each having a sectional area of $1\frac{1}{8}$ sq. in. = combined sectional area of $2\frac{1}{4}$ in. equal to a load of about 4 tons per sq. in.

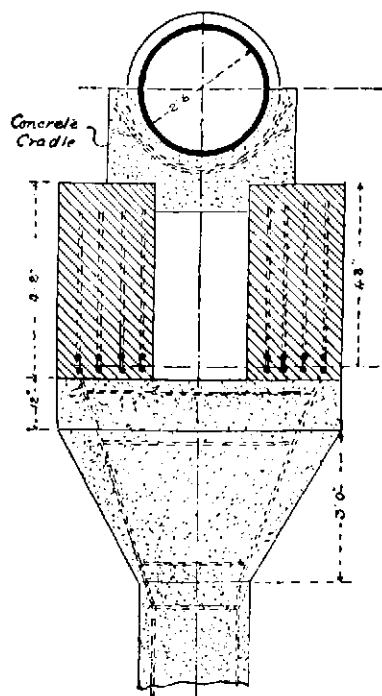


FIG. 954.

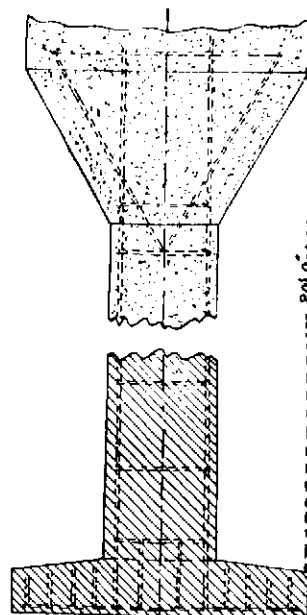


FIG. 955.

We proceed to consider a different form of construction which would be very adaptable to similar circumstances—namely, reinforced concrete.

Considering our 30-in. water main.

Span = 50 feet.

Load = 340 lbs. per foot-run.

Approximating the depth at half span we have say 48 in. = 4 ft. to centre of reinforcement. Referring to Chapter XX., the distance n to neutral axis =

$$\frac{Ecd}{t + E_c} = \frac{15 \times 600 \times 48}{17000 + (15 \times 600)} = 16.6.$$

$$\therefore \text{Approximate weight of beam} = \frac{25 \times 50 \times 50 \times 12 \times 150}{1728} = 65000 \text{ lbs.}$$

because we are taking the breadth as $\frac{d+2}{2} = \frac{48+2}{2} = 25$, and making $d = 48$.

Now, allowing for 2 in. of covering for the concrete, the beam will be $48 + 2 = 50$ in. deep $\times$ 25 in. wide. $\therefore$ the area of steel required will be, using Equation 255,

$$0.06bd = 0.06 \times 50 \times 25 = 0.06 \times 1250 = 7.500 \text{ sq. in.,}$$

which will be provided for by eight $1\frac{1}{8}$ -in. diameter rods.

Our beam will therefore be 50 in. $\times$ 25 in., and the reinforcement will be

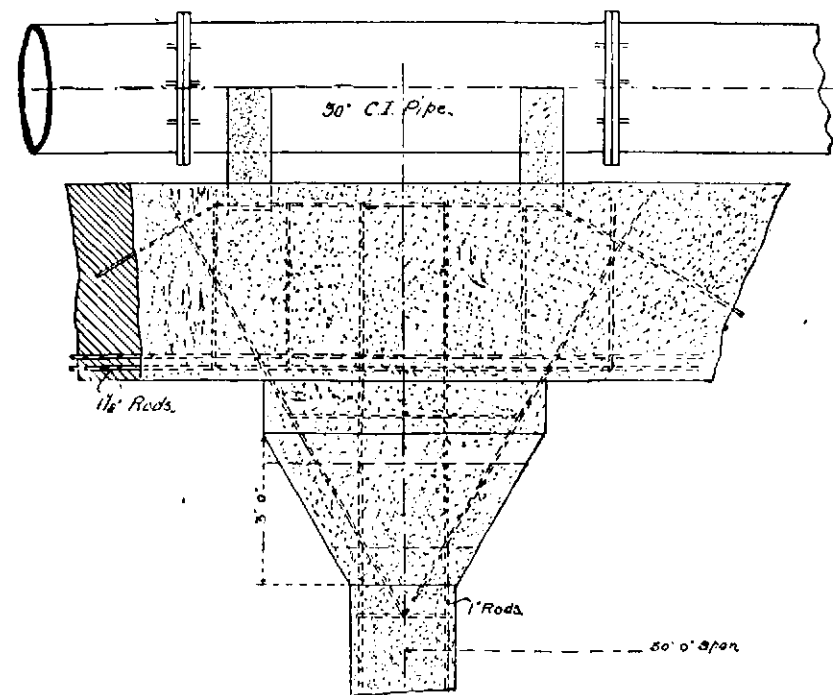


FIG. 956.

8 rods $1\frac{1}{8}$ -in. diam. The total load to be supported by the piers = $120,000 + 17,000 = 137,000$ lbs., of which 68,500 are supported by the central pier, which divides the total span of 100 feet into two of 50 feet. $\therefore$ a pier to stand a load of, say, 31 tons at 20 feet high, would be made, say, 2 ft. square = 4 sq. ft. of sectional area, which gives a stress intensity of 7.8 tons per sq. ft. It would be reinforced with four 1-in. diam. rods, as shown in figs. 954, 955, and 956.

We will now consider the method of designing tanks of steel and cast iron of the type which would be used for storing comparatively small quantities of water, in a more or less elevated position, so as to give the required delivery-head. Such tanks would find place in waterworks schemes, both gravitation and pumping, for domestic supply, tanks for such duties as storing water for power-houses, mills, etc., either for feed or circulating purposes, tanks for supplying water cranes at railway stations, oil tanks, and other purposes too numerous to mention.

The methods of designing such works economically, and successfully carrying them out, is not so much understood by engineers, especially younger men and pupils, for whom this book is essentially written, because the work is, more often than not, left in the hands of specialists, who are only too glad to execute designs with the hopes of getting the work. At the same time it is necessary, in these days of keen competition, that the engineer himself should have some knowledge of the work, so as to be capable of carefully scrutinising and advising his clients regarding the tenders.

The principal calculations for the work are based on the simple theorem of applied mechanics, which states that the pressure per unit area on the sides of a tank is half the pressure per unit area on the base, and that this pressure has a certain point of application which is termed the "centre of pressure," and which is found at one-third the depth from base to top water-level.

Let us consider circular tanks first, having flat bottoms. These would usually be supported on a system of steel joists, and it is particularly necessary to observe that the longitudinal joints in the bottom plates are in no case parallel to these joists. The plates should be of steel, having a factor of safety of at least 4, and they should be specified with a tensile stress of not less than 28 and not more than 32 tons per sq. in. and an elongation of 20 per cent. in 8 in. A very common error which the designer must guard against is so drawing out the plates that when these are riveted together the caulker has to work left-handed, a great disadvantage to his carrying out the work properly. Besides flat bottoms, circular tanks are frequently made with dished, or even coned, bottoms; the advantage of this mode of construction obviously being that a system of cross joists is thereby done away with, the base being self-supporting, the tank merely resting on a circular ring or curb. These dished bottoms, however, should have a curvature of not less than the tank diameter in order to be efficient. These tanks, when applied to domestic supplies, have, more often than not, a central shaft having a spiral stairway for entry into the tank for cleaning and inspection; while inlet and outlet pipes enter into the base of the tank by means of telescopic expansion joints, to cope with the continually altering sag on the bottom caused by the varying head of water above them.

A notable example of this class of work is the one at Norton in Cheshire. It belongs to the Liverpool Corporation. Its capacity is stated by Messrs Tudabery & Brightmore, in their *Waterworks Engineering*, to be 600,000 gallons and 76 ft. in diameter. Having settled upon the radius of the tank = R_1 , we can make the radius of the bottom = $2R_1 = R_2$.

Assume we require to find the size of plates for a tank 50 ft. in diameter. Therefore $R_1 = 25$ ft. and $R_2 = 50$ ft.; then the thickness of the bottom plates is obtained by the formula—

$$T = \frac{R_2 H w}{24 P \times .5} + .25 \quad (494)$$

where T = thickness in inches.

H = depth of water in feet.

P = safe stress on plates, say 6 tons per sq. in.

w = weight of 1 cub. ft. of water in tons = $\frac{1}{25}$.

$\therefore T = \frac{50 \times 20 \times \frac{1}{25}}{24 \times 6 \times .5} + \frac{1}{4} = \frac{4}{3} + \frac{1}{4} = \text{say } .4 + .25 = .65$, say $\frac{5}{8}$ in. plates were used.

Now regarding the side plates, the pressure will decrease uniformly to nil at top water-level. Therefore in deep tanks it would be economical to use two sizes of plates. For the tank in question, the greatest thickness of side plates required is given by the formula—

$$T = \frac{R_1 H w}{12.0 P \times .5} + .25 = \frac{25 \times 20 \times \frac{1}{25}}{12.0 \times 6 \times .5} + .25 \quad (495)$$

$$= \text{say } \frac{4}{3} + \frac{1}{4} = \frac{17}{12} = 6.$$

Probably $\frac{5}{8}$ -in. plates would be used. But substituting 10 for 20 as value of H , we find $\frac{5}{8}$ -in. plates would do; which means to say, above 10 ft. in depth we could safely use $\frac{5}{8}$ -in. plates with advantage in economy.

The details of such a tank are shown by figs. 957 and 958, fig. 957 showing a section of this tank, and fig. 958 an enlarged detail of the joints,

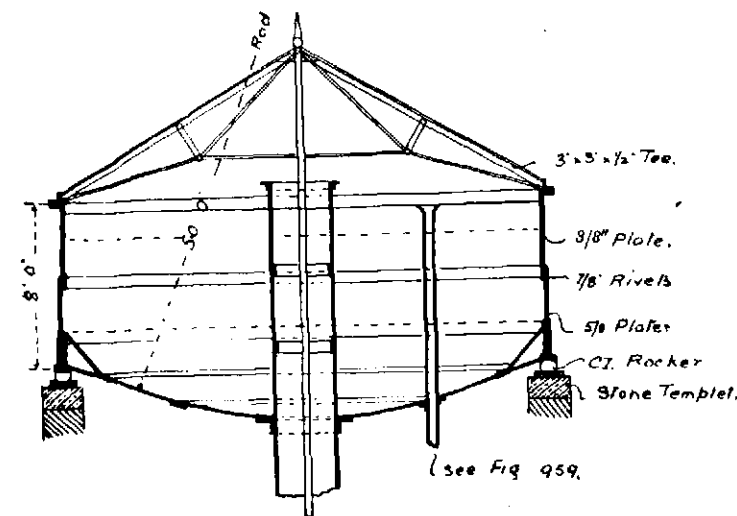


FIG. 957.

while fig. 959 shows details of an expansion joint for the inlet or outlet pipes. For the circular seams single-riveted lap joints will do well, while for radial seams butt joints with cover strips should be provided for. The above equations assume a joint efficiency of .5. Particular attention is drawn to the stout rocker rings which these tanks rest on (fig. 958). It is to counteract movement due to temperature changes. When we have a tank with a conical bottom the value of R_2 is taken perpendicular to the bottom plates, as shown by fig. 960.

With flat-bottomed tanks the size of the bottom plates would naturally be controlled by the pitch of the supporting joists, which will vary with different cases, and is best left to the discretion of the engineer: 8 ft. apart is about a maximum. The plates would probably be made the same thickness as the side ones, and the joists spaced accordingly.

Fig. 961 shows a plan of a circular tank of 25 ft. in diameter, and fig. 962 shows an enlarged detail of the bottom and sides joint, which requires particular attention. The angles used would be $\frac{1}{8}$ thicker than the bottom

$\frac{7}{16}$ plates would be used. Again, if d = drain of rivets used in inches,

$$d = 1.2 \sqrt{T} \quad (498)$$

$$\therefore d = 1.2 \sqrt{\frac{7}{16}} = 1.2 \times .66 = .792.$$

$\frac{7}{8}$ rivets would be used; and if p = pitch of rivets,

$$p = 3d + T \quad (499)$$

$$= (3 \times \frac{7}{8}) + \frac{7}{16} = 2\frac{5}{8} + \frac{7}{16} = 3\frac{1}{8}, \text{ say } 3 \text{ in.}$$

The thickness of butt covers should be $1\frac{1}{8}T$ (500)

$$= \frac{7}{16} \times \frac{9}{8} = \frac{1}{2}, \text{ say } \frac{1}{2} \text{ in. plates are used,}$$

while their width $= 6d + \frac{1}{2}$ (501)

$$= (6 \times \frac{7}{8}) + \frac{1}{2} = 5\frac{3}{4} \text{ in.}$$

The stays now demand attention. They are being placed 4 ft. apart. Area held by each stay = $4 \times 7 = 28$ ft. The stays should have a factor of safety of 8. Allowing a maximum stress of 5000 lbs. per sq. in., total pressure on sides

$$= \frac{HAW}{2} = \frac{7 \times 27.25 \times 10 \times 62.5}{2} = 59609 \text{ lbs.}$$

This will be held by 6 stays, each stay supporting

$$9935 \text{ lbs.} \div 5000 = 1.987 \text{ sq. in., say } 2 \text{ sq. in.}$$

But we have been assuming the use of direct stays, in place of which we are using diagonal stays. Call the area of direct stay A_1 , and of the diagonal stay A_2 . Referring to fig. 964,

$$A_2 = A_1 \frac{L}{D} \quad (502)$$

And if we settle D , the correct dimension for L ,

$$L = \frac{A_1^2 \times D}{A_2} = \frac{2 \times 2 \times 3}{2} = 6 \text{ ft.}$$

$$\therefore A_2 = 2 \times \frac{4}{3} = 4 \text{ sq. in.}$$

Therefore we would use $5\frac{1}{2} \times \frac{3}{4}$ flats, say, and, as the stays on the ends are spaced the same distance apart, we would use the same dimensions. By working out simple calculations, we find the weight of the tank = about 10 tons, and of water contained therein $51\frac{1}{2}$ tons; say total weight = 62 tons. The main joists, seven in number, each having a span of, say, 8 ft., must bear $\frac{62}{7}$, say 9 tons each. Looking up a table of joists, we find 8 in. $\times$ 4 in. $\times$ 25 lbs. will do. Again, we have four bearing joists, each having a load of, say, $2\frac{1}{2}$ tons over a span of 4 ft. 3 in., for which we will use 3 in. $\times$ 2 in. I beams.

Again, the load of 63 tons is supported by 14 piers, each pier having to bear nearly 5 tons over a sectional area of

$$\frac{15 \times 15}{144} = \frac{225}{144} = 1.57 \text{ sq. ft.}$$

and $\frac{5}{1.57}$ = say 3.20 tons per sq. ft., which are spread out in the foundations to give a bearing on the soil of less than 1 ton per sq. ft.

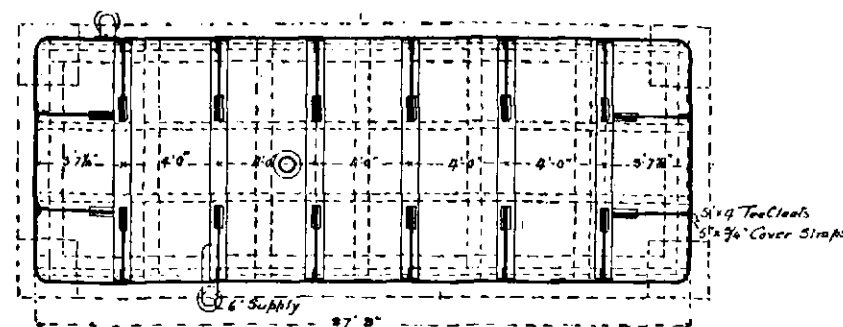


FIG. 965.

At the corners would be gusset stays as shown. Figs. 965 to 971 give full details of the tank as worked out.

We must now turn our attention to tanks of cast iron. Although these tanks are losing popularity, yet they still demand attention. They are made up of plates of rectangular section usually, which are bolted together, those at

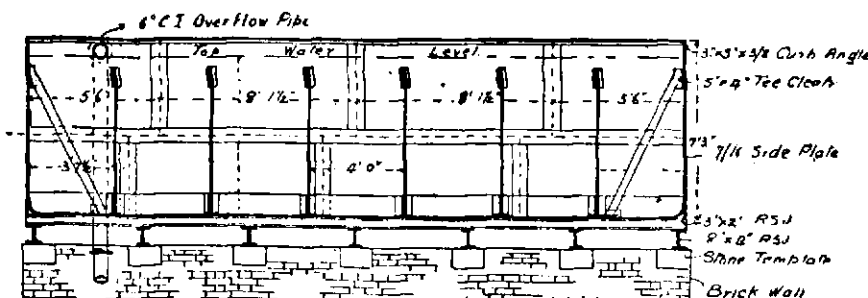


FIG. 966.

the edges and corners being specially shaped. For erection in country districts cast iron, no doubt, has great advantages, especially in the absence of skilled platers and riveters. Like steel tanks, the shape may be rectangular or circular, with flat or coned bottoms, the latter construction being somewhat unusual. Let us take a numerical example. A cast-iron tank is required—size, 20 ft. $\times$ 20 ft. $\times$ 10 ft. Let P = pressure due to maximum head of water in pounds per sq. in. = $434 \times 10 = 4340$ lbs. per sq. in. Say the size of the side plates is to be 3 ft. $\times$ 3 ft.

plates, and a width of flange = 8 times the thickness. Fig. 963 shows the usual design of roof for circular tanks, having a rise of $\frac{1}{10}$ the diameter usually. Knowing this, we want to find the correct radius to strike it to, in which case we use the equation—

$$R = \frac{S^2}{8r} + \frac{r}{2} \quad (496)$$

R being the radius, r = rise, and S = span, all in the same units of measure. The scantlings given would suit a roof up to 40-ft. span. The roof terminates in a circular cap plate, the other plates being specially laid out to templet, and, according to their position, are known as the top strake, middle strake, and bottom strake. The covering would be $\frac{1}{4}$ -in. or $\frac{5}{16}$ -in. plates according to circumstances. Round the eaves there should be 5 in. or 6 in. of projection to throw off rain clear of the tank body. For domestic supplies the roofs are often made double to keep the supply cool in hot weather. For

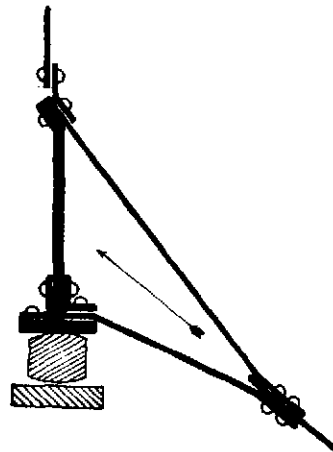


FIG. 958.

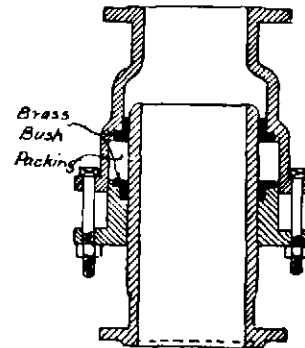


FIG. 959.

the same reason, the body of the tank is sometimes lagged with wood and hair felt. Sometimes the interior is lined with asphalt.

We now come to consider tanks of the rectangular type. In this work the joints should be butt joints, having a single cover strap inside, and then joints between the side and bottom plates are preferably made by flanging in this type of tank, the flanges having an easy radius of at least 5 in. The vertical corners may be done in the same way. Regarding the shape of the tank, the square is that geometrical figure which contains the largest area for the least perimeter, but it is not always policy to adopt this course: 6 ft. to 7 ft. is the most economical depth. The supporting should consist of a system of upper and lower joists, the upper ones being placed 3 ft. centre to centre. When designing, it is advisable, where possible, to avoid seams at the vertical corners. An angle iron would be run along the top to stiffen the sides, and if there is a roof it will also assist in supporting it. Stays would be placed at every cover strap—viz. 4 ft. apart c. to c., as plates are usually rolled 4 ft. wide. The angle irons holding these stays should be hard down on the horizontal butt covers. It is economy in rectangular tanks (whose depth does, as stated, not usually exceed 6 ft. to 7 ft.) to use plates of the

same thickness throughout. We will now work out the design for a steel tank in detail.

Capacity, 11,500 gallons; shape, rectangular.

The shape, we will suppose, is governed by some existing circumstances to 27 ft. 3 in. $\times$ 10 ft.

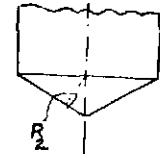


FIG. 960.

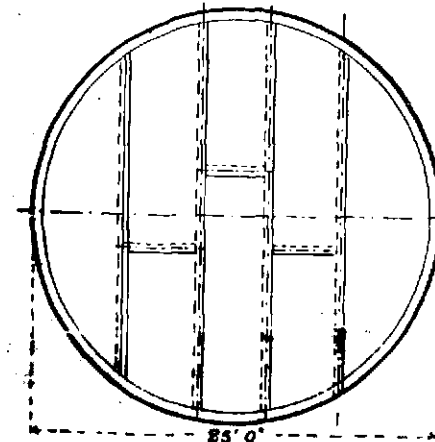


FIG. 961.

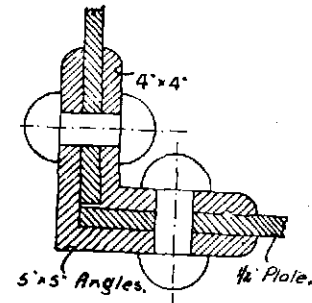


FIG. 962.

Now
and

$$11,500 \text{ gallons} = 1840 \text{ cub. ft.},$$

$$10 \times 27 \cdot 25 = 272 \cdot 5 \text{ sq. ft.}$$

$$\therefore 1840 \div 272 \cdot 5 = 6 \text{ ft. } 9 \text{ in.},$$

and allowing 6 in. above top water-level, which is usual, the size of our tank will be 27 ft. 3 in. $\times$ 10 ft. $\times$ 7 ft. 3 in. The first thing to do is to settle

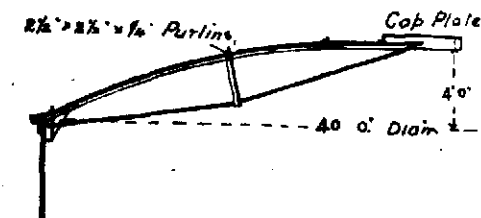


FIG. 963.

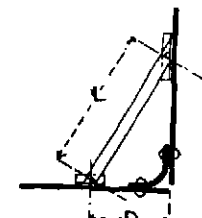


FIG. 964.

the pitch of the small joists, say 3 ft. = P , while maximum depth of liquid = 6 ft. 9 in. = D , which, allowing for contingencies, we will assume to be 7 ft. Now, knowing P and D , and allowing a factor of safety of 5, if T = thickness of bottom plates,

$$T = \frac{P \sqrt{D}}{18} \quad (497)$$

$$= \frac{3 \times \sqrt{7 \cdot 0}}{18} = \frac{7 \cdot 92}{18} = \cdot 44 \text{ in.}$$

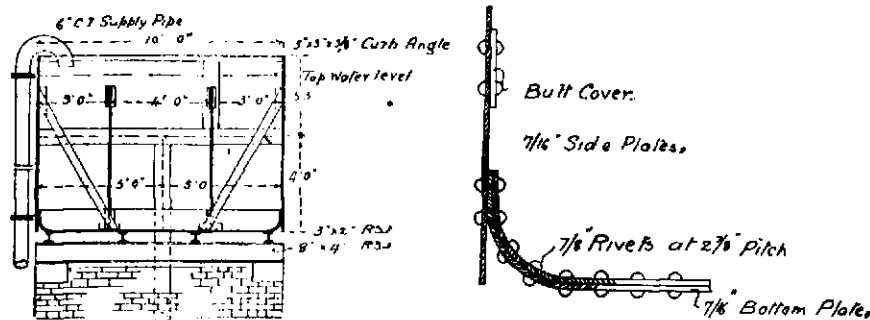


FIG. 967.

FIG. 968.—Detail of Bottom and Side Joint.

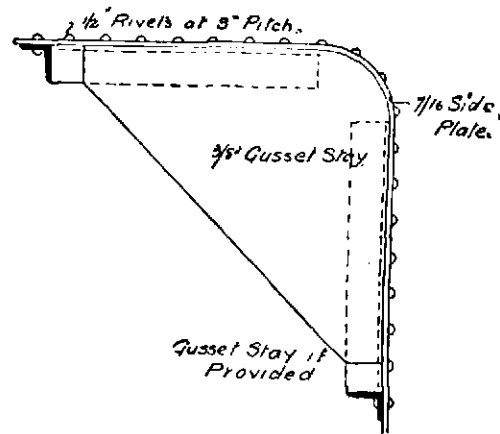


FIG. 969.

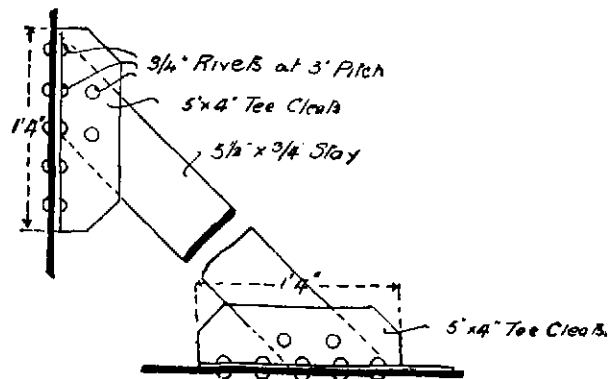


FIG. 970.

Let t = thickness of plate in ins.
 Let b and d = dimensions of plate = 36 in. and 36 in.
 Let P = total pressure on each plate = plh .

$$\text{Then } t = .01b \sqrt{\frac{P/b}{t^4 + b^4} + \frac{1}{4}} \quad (503)$$

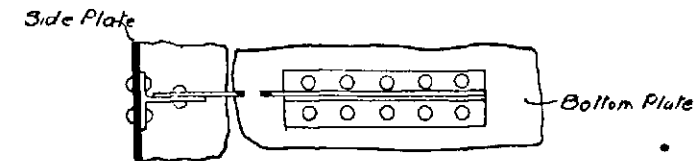
(Molesworth's Pocket Book.)

In which case,

$$t = .01 \times 36 \sqrt{\frac{4.34 \times 36 \times 36 \times 36 \times 36}{(46656 \times 36) + (46656 \times 36)} + \frac{1}{4}}$$

$$= .36 \sqrt{\frac{4.34}{2}} = .36 \sqrt{2.17} = .36 \times 1.5 = .540 + .75.$$

The plates would probably be $1\frac{1}{4}$ in. thick.



Detail of Slays,

FIG. 971.

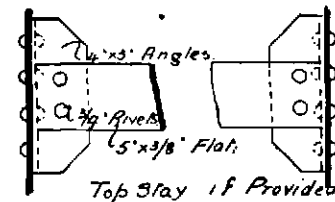


FIG. 971A.

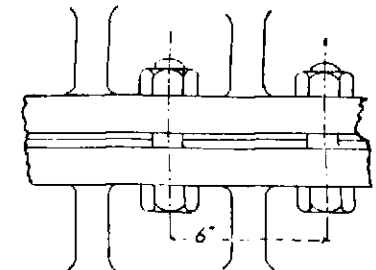


FIG. 972.

Now we make the flanges $\frac{1}{2}$ in. thicker and in depth three times the thickness, in which case we shall have flanges $1\frac{1}{2}$ in. thick and $4\frac{1}{2}$ in. deep.*

Details of the plate as cast are given in figs. 972 and 973. The faces are usually planed and put together with a little red lead, or rust joints may be formed. The bolts used are usually same diameter as the thickness of flange. The bottom plates would be the same thickness, as the side ones are calculated on the maximum pressure.

Let us take now the case of a circular tank having a conical bottom, supported on a brick wall around the outer edge and having a stand-pipe in the centre, as shown diagrammatically in fig. 974.

* The example given here is merely numerical to illustrate the text. The common engineering practice is to make cast-iron tanks 4 ft. 9 in. deep, which will give a plate thickness of about $\frac{1}{2}$ in. only. It remains with the engineer to use his judgment and experience in economics to find which depth gives the most economic use of the material at his disposal. This may necessitate increasing the value of the depth of flange.

Let the angle which the floor makes with horizontal $= 35^\circ = \theta$, and let the tank be 20 ft. in diameter and 10 ft. deep as shown.

Let r = radius of bottom = 120 in.

Let f = 3000 lbs. per sq. in. safe stress.

Then
$$t = \frac{pr}{2f \sin \theta} \quad (504)$$

($p = 4.34$, $\sin \theta = .57$),

$$\frac{4.34 \times 120}{2 \times 3000 \times .57} = \frac{2.17}{14.25}$$

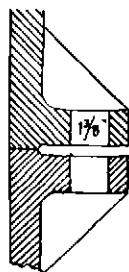


Fig. 973.

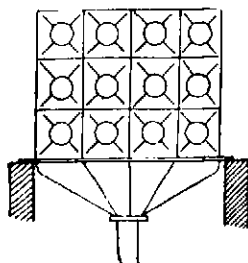


Fig. 974.

$= .152$, a value too small for practical castings: $\frac{3}{8}$ -in. plates at least would be used, the side plates being the same, although on large tanks the stress on the side plates may exceed that on a conical bottom. The bottom plates should not be cast in sections much over 2 cwt. They would appear on plan as in

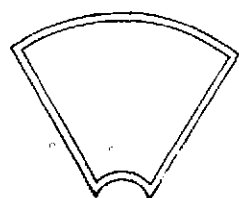


Fig. 975.

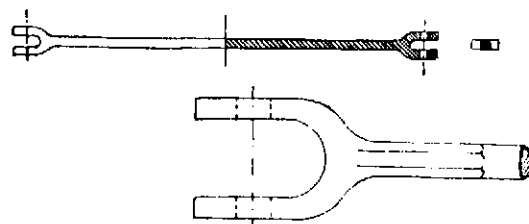


Fig. 976.

fig. 975. The pitch of bolts in cast-iron tanks is usually 6 in. Now on the cast-iron curb there will be a certain pull due to the conical bottom which will set up compression therein. First of all, find the weight of water in the tank,

$$20 \times 20 \times .785 \times 10 = 3140 \text{ cub. ft.,}$$

neglecting slope of bottom.

Let K = compression set up in curb;

its value
$$= .159 W \cot \theta \quad (505)$$

$$= .159 \times 3140 \times 62.5 \times 1.42$$

= about 20 tons:

say, for sake of uniformity, we use $\frac{3}{8}$ -in. plates, we have $20 \times \pi \times \frac{3}{8}$ = nearly 40 sq. in. of metal, giving a stress of about $\frac{1}{2}$ ton per sq. in.

Cast-iron tanks are fitted together with the flanges inside, and are not usually in practice more than 5 ft. deep when rectangular. Stays are usually provided (fig. 976). The supporting joists would be placed as far as possible underneath the joints so as to relieve them of stress. The appurtenances of tanks are not numerous.

The supply pipe, if from pumping engines, should enter the tank as shown by fig. 967; when from a gravitation supply, a ball valve of the simplest mechanical construction is necessary. The overflow should be at least a quarter as much again in diameter as the supply, while the outlet should be bell-mouthed in shape and have a copper straining rose thereon. The usual valves are provided. In addition to these a depth indicator is essential. It should be as simple as possible. Details of a simple piece of apparatus appear in figs. 977 and 978.

The reader is referred to fig. 958. A large arrow will be observed. It is to denote the direction of the bottom plate when produced, which must pass through the centre of gravity of the triangular space enclosed by the plates as shown.

The actual staving of tanks is a matter which demands more attention than has hitherto been given to it. As we have before pointed out, the usual practice is to place stays near to the top of the stiffener, the remainder being 4 ft. to 5 ft. apart. The area (and hence pressure) supported by each stay is then calculated, while the stiffeners are designed to resist the bending moment set up by the water pressure. It is also a common practice to cut down the theoretical number of stays and allow a considerable amount of bulging to take place in the plates, setting up considerable tension therein, but this must not be so great that the effects appear unsightly, even if quite safe.

Let us take a practical example.

A rectangular tank 20 ft. $\times$ 30 ft. $\times$ 12 in. deep. The stiffeners are to be 4 ft. centre to centre. Hence the stays will be likewise. Allowing a factor of safety of 8 (to allow for corrosion), we have 4 tons per sq. in. as safe stress. The area supported by each stay $= 4 \times 4 = 16$ sq. ft. The depth below the water = 7 ft. 6 in., 1 cub. ft. of water = 62.5 lbs. $\therefore$ load on stay

$$= \frac{16 \times 62.5 \times 7.5}{2240} = 3.4 \text{ tons, say,}$$

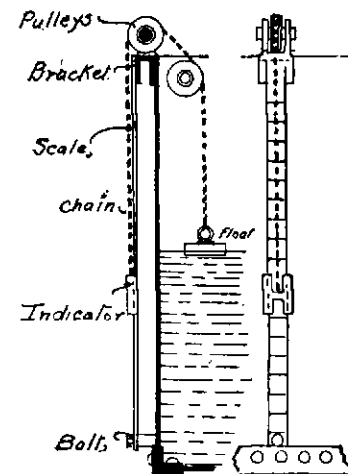
$$\therefore 3.4 \div 4 = .85 \text{ sq. in. section area, considering rivet holes, } \frac{3}{4} \text{ rivets.}$$

$$\text{One } 3 \text{ in. } \times \frac{3}{8} \text{ in. plate} = 1.13 \text{ sq. in.}$$

$$\text{One } \frac{3}{4} \text{-in. hole} = .28,$$

$$\therefore 1.13 - .28 = .85 \text{ sq. in. net sectional area, which is correct.}$$

The stays are all horizontal, the long ones being above the short ones, to take advantage of the consequent extra support to prevent any undue sagging.



Figs. 977 and 978.

Let the angle which the floor makes with horizontal $= 35^\circ = \theta$, and let the tank be 20 ft. in diameter and 10 ft. deep as shown.

Let r = radius of bottom = 120 in.

Let f = 3000 lbs. per sq. in. safe stress.

Then

$$t = \frac{pr}{2f \sin \theta} \quad (504)$$

($p = 4.34$, $\sin \theta = .57$),

$$\frac{4.34 \times 120}{2 \times 3000 \times .57} = \frac{2.17}{14.25}$$

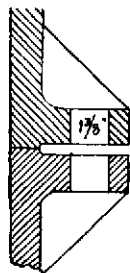


FIG. 973.

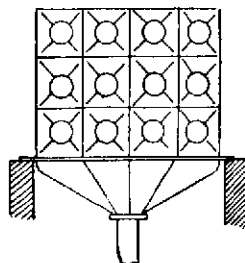


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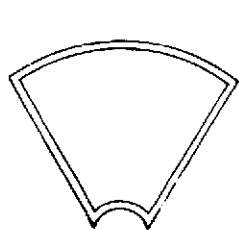


FIG. 975.

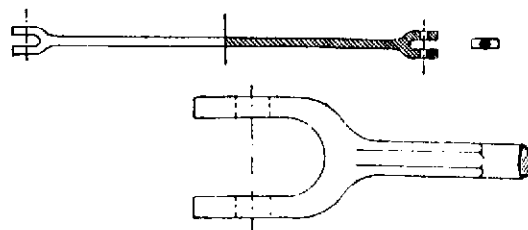


FIG. 976.

fig. 975. The pitch of bolts in cast-iron tanks is usually 6 in. Now on the cast-iron curb there will be a certain pull due to the conical bottom which will set up compression therein. First of all, find the weight of water in the tank,

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neglecting slope of bottom.

Let
its value

$$\begin{aligned} K &= \text{compression set up in curb;} \\ &= .159 W \cotan \theta \quad (505) \\ &= .159 \times 3140 \times 62.5 \times 1.42 \\ &= \text{about 20 tons;} \end{aligned}$$

say, for sake of uniformity, we use $\frac{5}{8}$ -in. plates, we have $20 \times \pi \times \frac{5}{8} = \text{about } 39 \text{ sq. in. of metal, giving a stress of about } \frac{1}{2} \text{ ton per sq. in.}$

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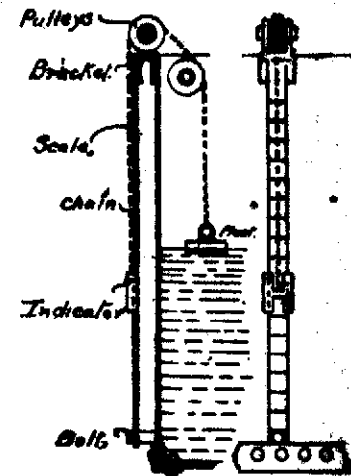
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$$= \frac{16 \times 62.5 \times 7.5}{2240} = 3.4 \text{ tons, say,}$$



FIGS. 977 and 978.

treated in books on Building Construction. The most usual are slates and corrugated iron; the fixing is effected in a variety of ways, as shown in our examples.

In this chapter, however, we mainly want to place before the reader the heavier class of structural engineering, leaving him to pursue the study of the more elementary details from other books. Much is to be learnt, however, in all kinds of this work from practical drawings, and I have included some of the latest examples of actual practice. The following examples were supplied to me by Messrs Dorman, Long & Co., Middlesbrough, whose experience in this class of work should prove of great value to the reader.

Fig. 979. Roof truss, 68 ft. span (centres of column), designed to carry 16 gauge galvanised sheeting, and to resist such wind pressure as occurs in this country. Trusses placed at about 12 ft. 6 in. centres.

Fig. 980. Truss 78 ft. 9 in. span for roof over auditorium of theatre. The covering of this roof was composed of slates upon boards, the purlins being of timber, and the trusses placed at 10 ft. 9 in. centres.

Fig. 981. Roof truss 108 ft. 2 in. spans for rolling mills. The covering was composed of boards, covered with tarred felt. Purlins were of 8 in. $\times$ 4 in. beams, with sag rods at intermediate points in their length, the trusses being 26 ft. 8 in. apart.

A roof for 40-ft. span, designed by the writer, is shown in fig. 982.

These drawings afford excellent examples of first-class drawing-office practice.

We now come to consider the last class of combined steel structure which concerns the municipal engineer—that of bridges for road traffic. We are not concerned here with heavy railway work, which purely belongs to the railway engineers and bridge designers. Of course the subject of bridge design is a very complex subject, and could not be fully dealt with in a book of this sort: for its complete understanding and appreciation a high mathematical knowledge is required. We have, however, pointed out the way in which we find the stresses in lattice girders and design them, and also how ordinary girders are designed. With the exception of arched ribs of steel, which often assume huge dimensions, bridges consist of a system of girders of various kinds, and the actual flooring, together with other minor details. It therefore remains for us in this chapter to discuss the way, not in which the actual girders are designed, but the way that they are put to practical use in facilitating transit.

Let us consider the flooring.

In railway bridges it is usual to have girders or other means of taking up direct stresses immediately under the rails at 4 ft. 8½ in. centre to centre (or according to the gauge). In road bridges, however, the whole of the floor must be capable of withstanding the maximum load upon it. And it must also be quite water-tight; hence we have to consider different classes of road metalling and paving.

Again, a former practice was to have self-supporting floors made of masonry or concrete on the arched principle, or reinforced concrete.

The general modern practice is, however, to use a special form of troughing filled with concrete, of which examples will appear later on. This troughing is now manufactured to standard sizes by steelmakers, who guarantee it to stand certain loads under fixed conditions. For ease of construction, lightness, durability, and economy, it readily presents itself to the municipal engineer for highway bridges. It will afford an absolutely water-tight superstructure

The next stay will have a load upon it equal to $\frac{16 \times 62.5 \times 3.5}{2240} = 1.5$ tons
 $\div 4 = .38$ sq. in. sectional area only required, for which we would use a
 2 in. $\times \frac{5}{16}$ flat, while the top stay would conveniently be made the same size.
 Each stay A should then have two $\frac{5}{16}$ -in. rivets, being attached to the stiffener
 B by gusset plates C (fig. 978b), while the total load on the portion between
 the bottom curb and bottom stay = $\frac{16 \times 9.5 \times 62.5}{2240} = 4.2$ tons.

We shall take the B.M. = $\frac{WL}{12}$ assuming them to be fixed.

W of course = load on stiffener, while L = the unsupported length only = 4 ft.

$$\therefore \text{B.M. in.-tons} = \frac{4.2 \times 48}{12} = 16.8.$$

Let us try a 4-in. $\times$ 3-in. $\times \frac{1}{2}$ -in. angle (as fig. 978a), and find its modulus,
 and also of this plate.

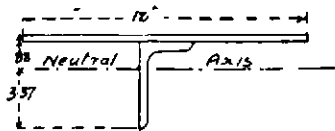


FIG. 978a.

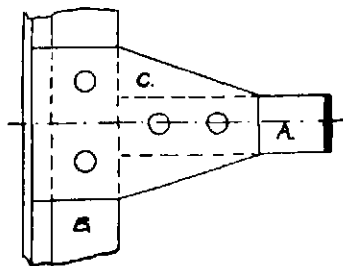


FIG. 978b.

Let their combined moments of inertia = 8.84 and the modulus = 2.6.
 $\therefore \frac{\text{B.M.}}{\text{modulus}} = \frac{16.8}{2.6} = 6.4$ tons per sq. in. The stiffener would be riveted to
 the side plate by $\frac{5}{16}$ rivets at 6-in. pitch, while the bottom curb would probably
 be 3-in. $\times$ 3-in. $\times \frac{5}{16}$ -in.

Now the next important type of structure which demands the reader's
 attention is that of roof trusses. Much has already been said on the theory
 of such trusses in Chapters X. and XVII., the latter bearing principally on
 wood trusses, while Chapter XII. deals with the method of calculating such
 trusses by means of stress diagrams. We proceed to deal here with points
 concerning the actual construction.

First of all we will deal with the effects of wind pressure.

Let P be its force in lbs. per sq. ft. (say 50).

Let α = angle which the roof surface makes with the direction of the wind,
 which is always considered horizontal.

Then we have three values of importance, viz. —

N the normal pressure.

V the vertical component.

H the horizontal component.

Then $N = P \sin \alpha \cdot 1.34 \cos \alpha - 1$ (506)

$V = P \cot \alpha \sin \alpha \cdot 1.84 \cos \alpha$ (507)

$H = P \sin \alpha \cdot 1.84 \cos \alpha$ (508)

Now, unlike other forms of structure, formulae for the accurate design of
 steel roofs are not numerous, accurate, or handy to use, hence the general
 practice is to resort to stress diagrams. These, when used for practical pur-
 poses, must be done to a large scale, on a large piece of double elephant draw-
 ing-paper, and a systematic manner of procedure must be maintained
 throughout; that is to say, on the same sheet must be neatly tabulated all
 stresses set up in each member, the required sectional areas, etc., and all
 details, together with a rigid system of lettering: without this, such diagrams,
 roughly done, are of no practical value to the designer.

Now, in roof design, there are certain fundamental rules to be observed.
 In the first place, engineers have found it not advisable for any principal
 rafter to be unsupported for more than 8 ft. Each joint must be so designed
 that it is in every way as strong as the members it connects, to avoid failure
 here. The limiting stress on the members is not usually more than 5 tons
 per sq. in. No welding should be permitted in any member. It will be
 observed that the tie-rods are usually cambered. The primary object is to
 reduce length of struts for economical reasons, but also to satisfy that
 perversity, human nature, in that most people would find a roof whose tie-rods
 had no camber unsightly; in the same way girders which are seen are usually
 cambered about $\frac{1}{4}$ in. to 1 ft. Why, however, a horizontal soffit to a roof or
 girder should be deemed unsightly the writer fails to see; suffice to say, it is
 so in popular estimation.

The gusset plates are usually $\frac{1}{2}$ in. thick. A rule exists which states that
 the combined thickness of two joint plates should provide a bearing surface
 for the rivets at least equal to that of the bar between them. Now the
 correct aim of a roof designer should be to have every member either in direct
 tension or compression. When, however, as occurs in small roofs, purlins are
 distributed along the length of a principal rafter, this does not become possible,
 and we have a transverse stress; if, however, the struts are fairly close, it will
 not affect the truss much more than causing the section of the principal rafter
 to be somewhat heavy. It is always an advantage to have short struts.

Cast iron should not be used in steel trusses. For large joints above 1 in.
 in diameter rivets are not advisable, and bolts are freely used. They are termed
 pin joints, and must be quite free in regard to the members being able to
 rotate about the bolt axis, but must be very well fitted to prevent any other
 looseness. Different sections will, of course, be used for the members accord-
 ing to circumstances. But in general practice, struts will be either angles,
 tees, tees back to back forming a + section channel iron, two channels so []
 kept apart by distance-pieces, and flat bar with distance-pieces to form a
 bulged strut. Cast-iron struts are not now generally used. For purlins,
 angles or tees are used. In joints, all members must be forged flat, and all
 widenings made of equal thickness, packing-pieces being used where required.
 The eyes are then all drilled at once. Principal rafters are usually of tee iron
 or two angles back to back, while ties are very often made of round bars.
 There is usually provided some means of tightening up these bars, and the
 generally adopted method is the turnbuckle shown in one of our beam
 examples. Unless the bars have a very large section compared to their load,
 plus threads must be formed for them to work on, so as not to materially
 weaken the rod. For tightening flats, etc., a jib-and-cotter arrangement
 is used.

The minor details of roofs, such as coverings, are things which we could
 not deal adequately with here: the subject, being elementary, is usually fully

for carrying road metal, and in most cases of its use cross girders can be dispensed with, and in some cases the main girders. A maximum amount of headway under the bridge is maintained.

The smaller sections of troughing are used for a variety of purposes in light structural work and flooring. Again, we have corrugated iron, which may be used in light and temporary work, and for heavier work Mallet's plate. Both these items have been already discussed.

Now, when despatched from the makers, troughing is usually riveted, as shown in fig. 983; the remaining riveting is done on the site, or bolts may

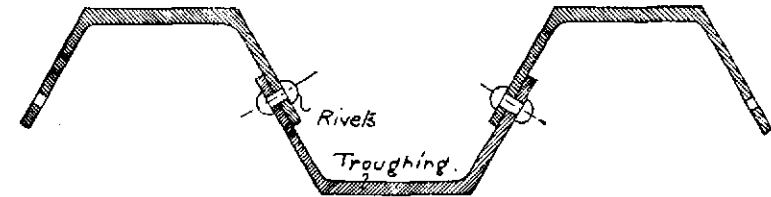


FIG. 983.

be used. Tables of safe loads will be usually found in makers' section-books. It may be well worth taking an example.

The roadway of a bridge is to be 21 ft. between the parapets. The maximum live load is a 16-ton traction engine and a 31-ton loaded waggon.

The dead load is calculated at 210 lbs. per sq. ft. The span of the bridge is 35 ft. It is required to construct it without main or cross girders.

Taking the load over $x x x$. We have total dead load, if $x=3$, of

$$\frac{35 \times 3 \times 3 \times 210}{2240} = 29.5 \text{ tons.}$$

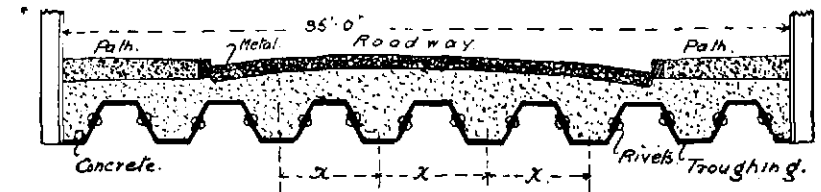


FIG. 984.

Now the bending moment due to the live load = 2390 in.-tons (nearly).

Max. B.M. due to dead load = $\frac{29.5 \times 35 \times 12}{8}$ = say 1580 in.-tons. $\therefore 2390 + 1580$
 = 3940.

Selecting a value for troughing from a catalogue, we find a type known as "E. Max." Its section modulus is given as 203.87×3 (for $x x x$) = 611.61.

$\therefore \frac{3940}{611.61}$ = a skin stress of 6.45 tons per sq. in., well within the Board of Trade limits.

For details of the above reasoning see fig. 984.

A roadway so formed can be metalled and paved the same as an ordinary road. Regarding the loads to take on a bridge, different designers have

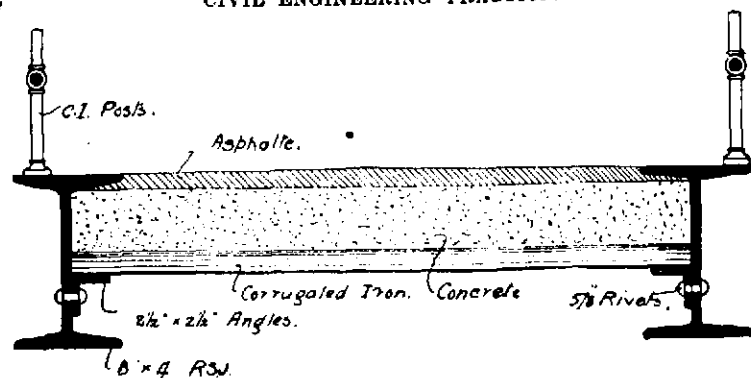


FIG. 985.

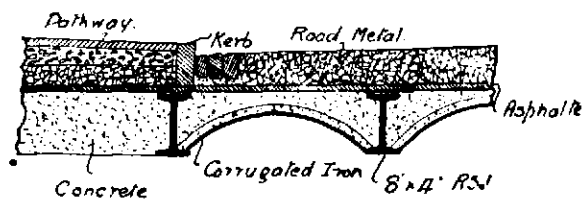


FIG. 986.

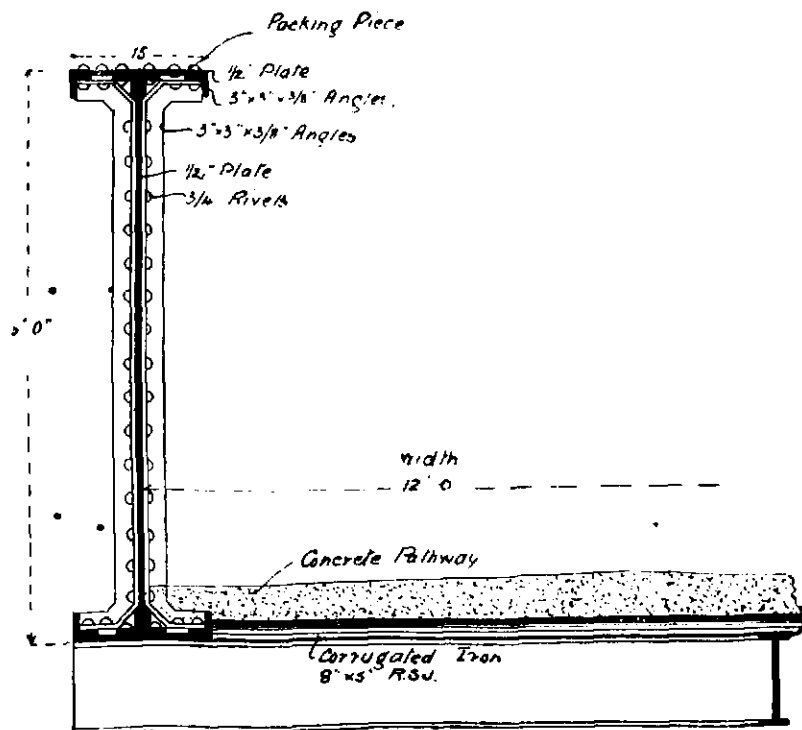


FIG. 986A.

different methods. For road bridges, however, as stated in our previous discussion of the subject, to which we refer the reader, we may always be safe in taking the maximum dead load at 120 lbs. per sq. ft. of roadway (+ pathway), and the maximum live load as anticipated under the circumstances. For a light footbridge, the construction shown in fig. 985, using corrugated iron, may be used. Another adoption of corrugated iron is shown in figs. 986 and 986A.

Now when we were discussing the subject of Stress Diagrams in reference to

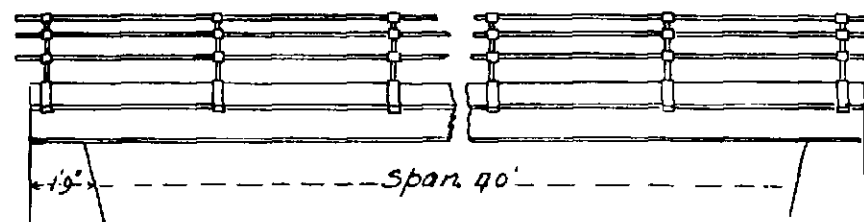


FIG. 987.

roofs, we made mention of roller bearings, for dealing with expansion. Large bridge trusses also have similar bearings, as will be shown in our further examples. They are usually placed at one end only, and when used they modify the stress diagrams, as already pointed out in Chapter XII. The actual design of such a roller is usually placed in the hands of specialists, but there is really nothing very complicated about them.

We have already said so much which bears upon the subject of bridges of the type here concerned in the previous chapter that it really only remains to give a few practical examples for the guidance of the reader.

Most of the following drawings, which represent the last words in bridge design, were supplied to me by Messrs Dorman, Long & Co., to whom I am greatly indebted.

Figs. 987 and 988. This is for a footbridge to be used in a country district, or positions where passenger traffic is not crowded.

Figs. 989, 990, and 991.

Also for a footbridge suitable

for carrying congested passenger traffic, live load per square foot provided for being 140 lbs.,—this load per square foot having been determined upon by the surveyor to the urban district council for which it was required.

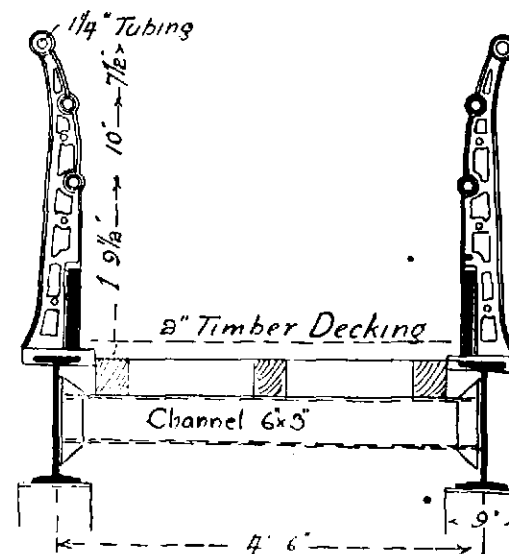
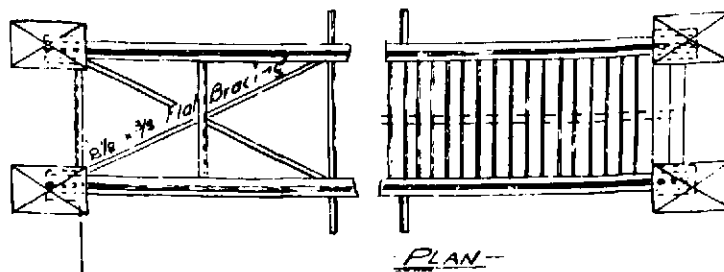
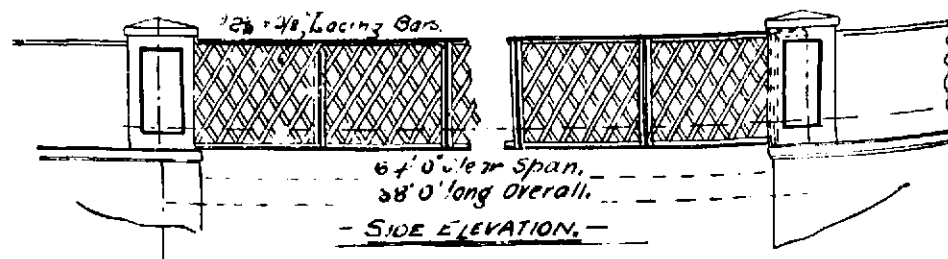


FIG. 988.



FIGS. 989 and 990.

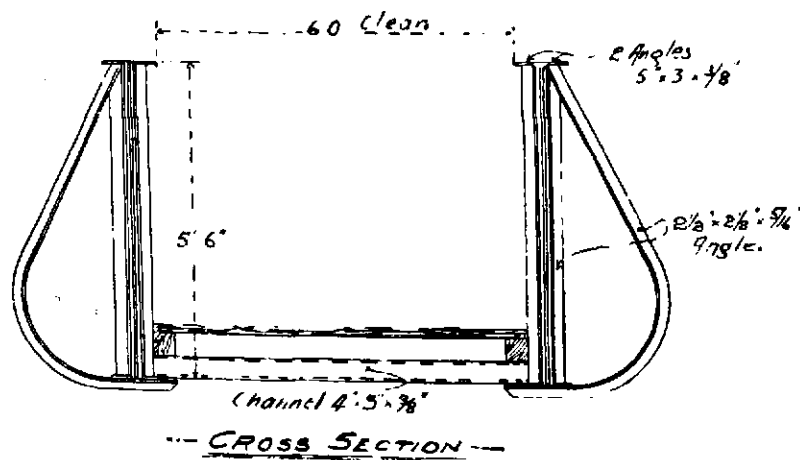


FIG. 991.

Figs. 992, 993, 994, 995, 996. This is designed as for a light road bridge for use in country districts, and is for a live load consisting of a 12-ton roller.

Figs. 997, 998, and 999. A road bridge, capable of carrying a 15-ton traction engine, or road roller about similar weight.

Figs. 1000, 1001, 1002, and 1003. Road bridge, capable of carrying a 15-ton traction engine hauling a trolley when fully loaded weighing 20 tons.

The span for this type of flooring should not be more than about 36 ft., and although the flooring in this instance consists of E troughing, viz. made up of troughs and flats, for smaller spans ordinary sections of troughing such as C and D might be used.

Figs. 1004 and 1005. Road bridge suitable for ordinary road traffic, designed to carry a 15-ton traction engine hauling a trolley weighing 20 tons when fully loaded.

Figs. 1006 and 1007. Road bridge in two spans suitable for carrying ordinary road traffic over a wide shallow stream, where it is possible to obtain a good foundation in the bed of the stream for the central pier. Designed to carry a 15-ton traction engine hauling a trolley 20 tons in weight.

Figs. 1008 and 1009. Road bridge designed to carry a 10-ton road roller.

By increasing size of girders at sides of bridges a stone or brick parapet may be used in place of steel hand-railing.

Figs. 1010, 1011, and 1012. Long span road bridge designed to carry a live load consisting of a 14-ton traction engine hauling a trolley when fully loaded weighing 20 tons. Also sufficiently strong to support a uniformly distributed load of 75 lbs. per sq. ft., thus providing for bridge being crowded with people.

Finally, owing to the courtesy of H. C. S. Ree, Esq., M.I.C.E., engineer to the Cardiff Railway Co., and Messrs Andrew Handyside & Sons of Derby, I am able to illustrate in figs. 1013 to 1017 a swing bridge, with the following particulars:—

Total weight of superstructure 254 tons, approximate total swinging weight 325 tons.

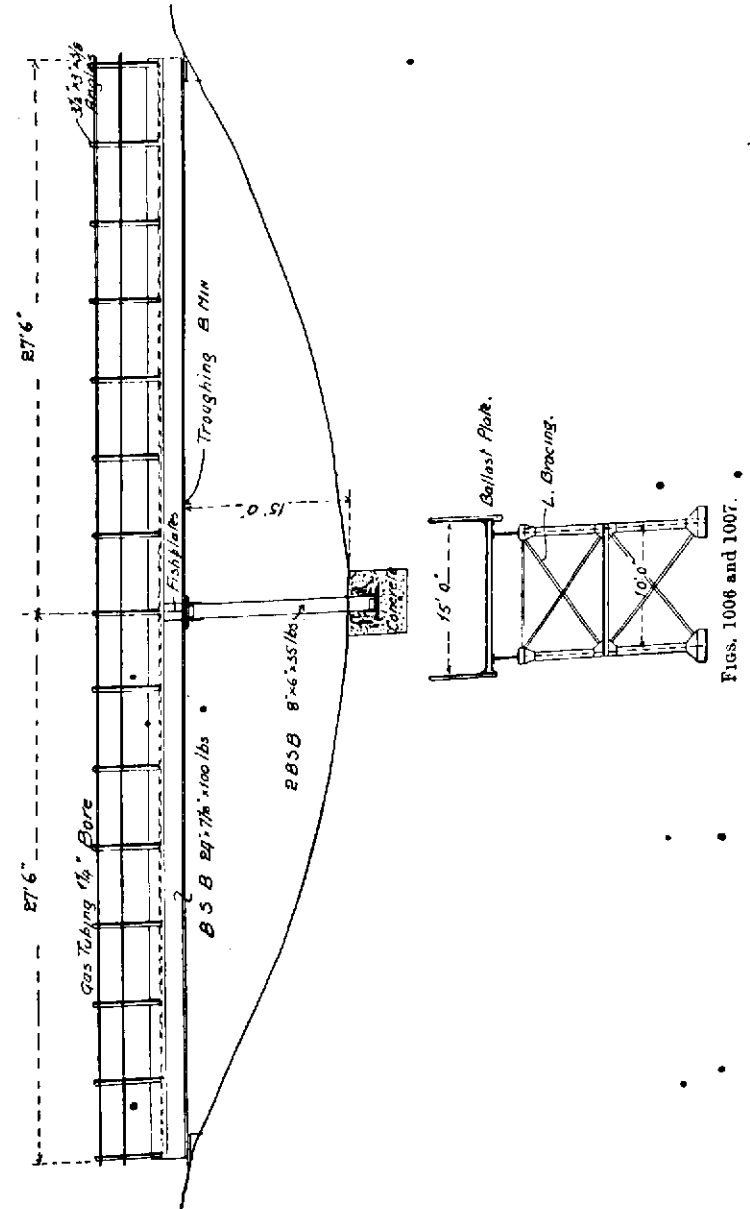
The bridge turns on a central hydraulic pivot with a balance roller at the rear end, this end being supported on wedge bearings, withdrawn by means of hydraulic lifting and lowering presses, so as to tilt nose end of girder clear of bearings before swinging. Hydraulic power supplied by horizontal rams on shore through a wire rope which turns a drum on the under side of the bridge. The hydraulic power plant was made by Messrs W. G. Armstrong, Whitworth & Co., Newcastle-on-Tyne.

In bridges which have straight bottom booms and curved top ones the stress diagrams, when drawn in accordance with the principles set forth in Chapter XII., become rather complicated and unsuitable.

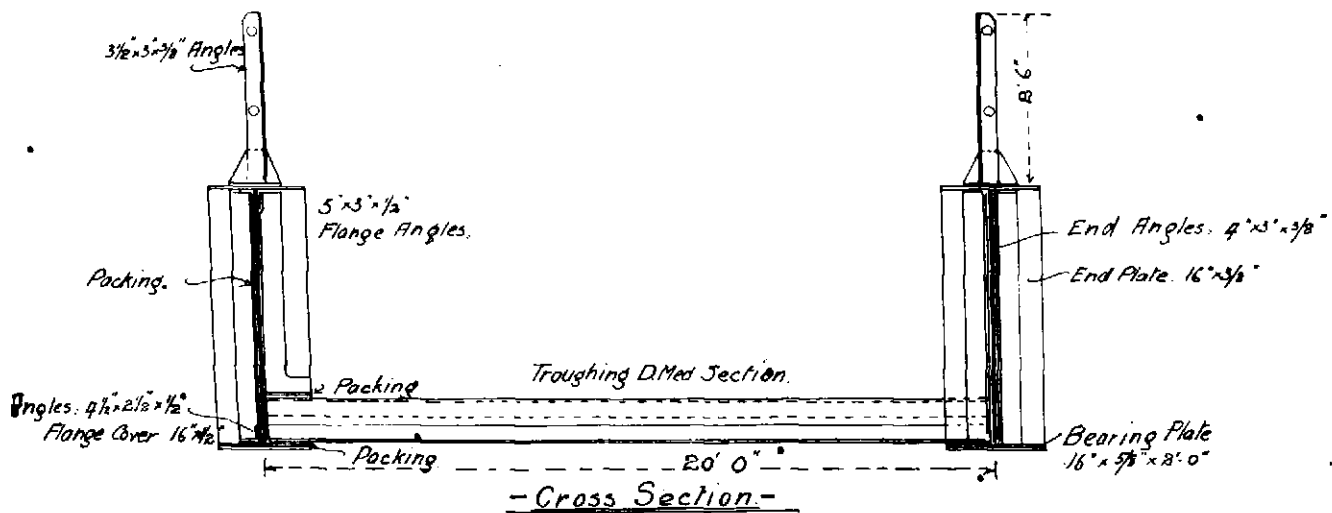
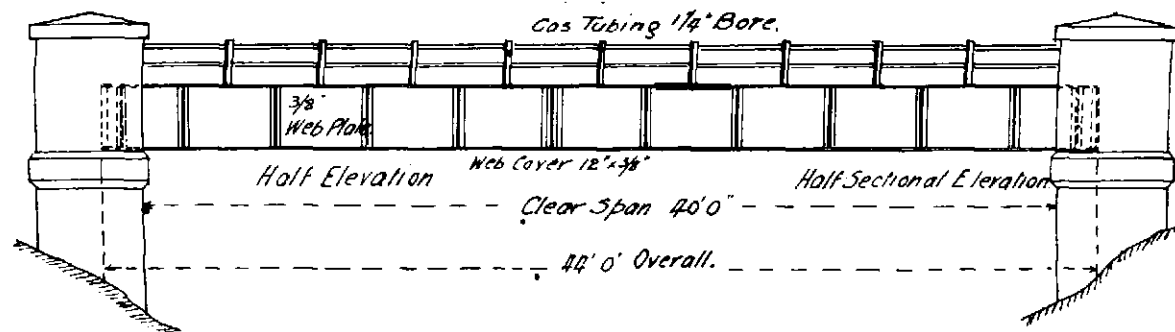
The matter, which is of course of importance to the bridge designer, is greatly simplified by a method introduced by an engineer, Mr Rinquist, whereby, by means of a few simple lines, all the stresses can be readily got at. The procedure is fully dealt with in Professor Jamieson's *Applied Mechanics*, to which the reader is referred, because, for obvious reasons, we do not discuss the matter here. It is also equally applicable to girders having both booms curved.

Previous mention has been made of the use of turnbuckles for tightening up tie-rods, provided that the design is properly carried out, and that the rods are thickened if necessary for plus threads; their efficiency is good. An alternative method of connecting tie-rods is by means of a sleeve and cotters,

as shown by figs. 1018 and 1019, which are subject to the following calculations:—



Let f_t = safe tensile strength of material employed, and f_s = safe shear
 $= .75f_t$



FIGS. 1004 AND 1005.

Let D = diameter of tie-rod.

" x = effective area of the cotter to take this load (double shear).

$$\therefore x = \frac{.7854 \times f_t D^2}{1.75 \times f_s} \quad (509)$$

$$= .5985 D^2 \text{ sq. in.}$$

Then if b = breadth of cotter along centre of rod and t = thickness of same, because $b \times t = .5985 D^2$, and assuming $b = 1.25 D$,

$$t = \frac{.5985 D^2}{1.25 D} \quad (510)$$

$$= .48 D.$$

The rod must be enlarged at the end to a diameter D_y . Call it y in formulæ.

$$\therefore y = \left(\frac{D}{2}\right)^2 \times \pi.$$

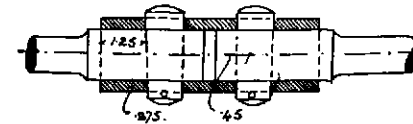


FIG. 1018.

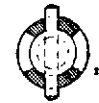


FIG. 1019.

$$\therefore \frac{3.141 y^2}{4} - y \times .4787 D = \frac{3.141 D^2}{4}.$$

$$\text{Now } D^2 = y^2 - \frac{y \times .4787 D \times 4}{3.141}.$$

$$\therefore y^2 - y \times .6095 D = D^2. \quad \therefore y = 1.35 D.$$

Then we have a certain correct length which this thick part should be continued beyond the cotter = Z ins. And

$$Z = \frac{.7854 \times f_t \times D^2}{1.35 D \times 1.75 \times f_s} \quad (511)$$

$$= \text{say, } .45 D,$$

while for the correct sectional area of sleeve immediately behind the cotter we have

$$a = \frac{.7854 D^2 \times f_t}{1.75 \times 1.25 D \times 1.75 \times f_s} \quad (512)$$

= say $.275 D$ as thickness of sleeve.

This last calculation assumes that the sleeve = $1.25 D$ long, double shear, and that the effective thickness is 1.75 times the actual thickness.

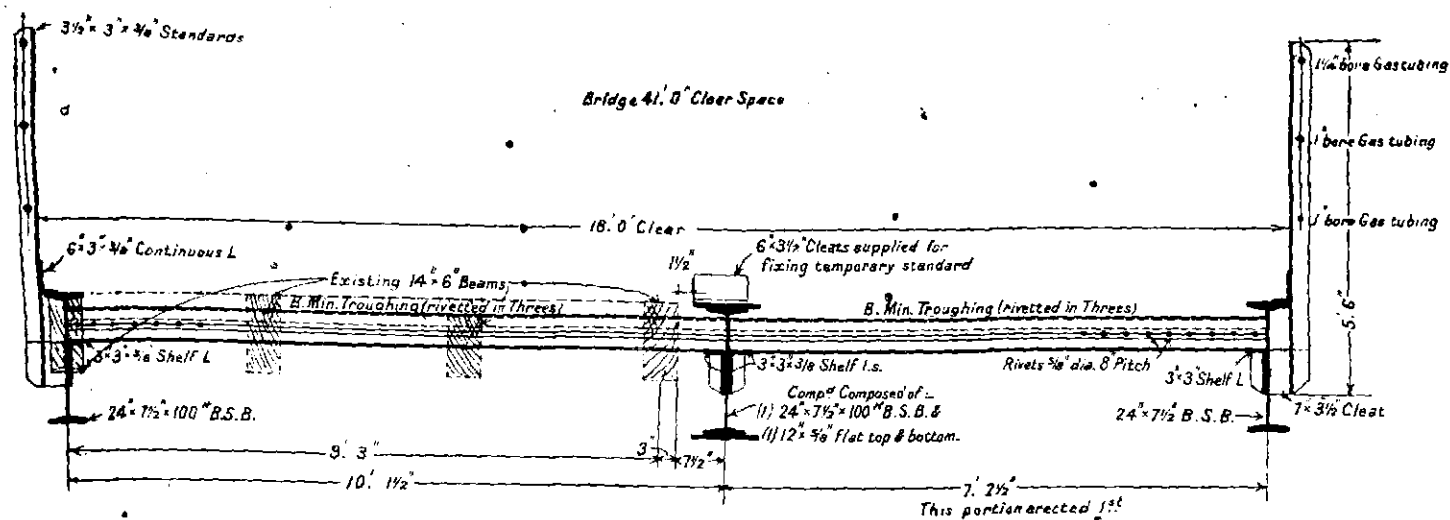
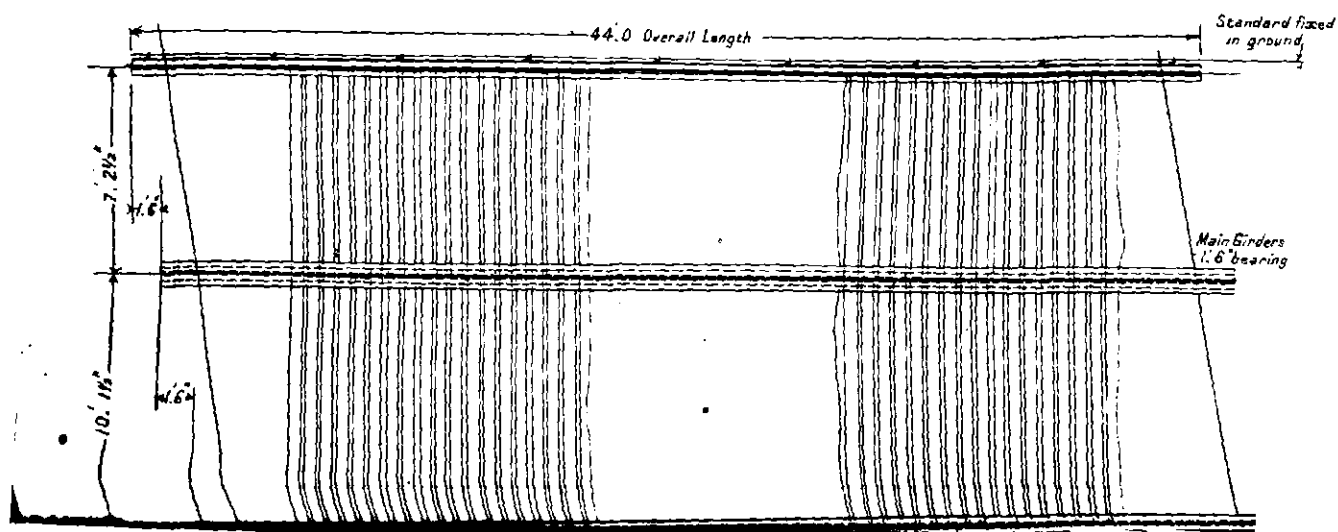


FIG. 1008.



CHAPTER XXXIX.

SPECIFICATIONS.

IN order to write a thoroughly sound and practical specification for a piece of engineering work it requires no small amount of literary skill on the part of the writer, besides a good deal of practical experience in construction work in general. To actually put on paper how to write a specification would be almost impossible; the art is only learned by actual dealing with and preparing such documents. With the object, however, of helping those whose experience in such work is limited, the writer has prepared the following notes as a guide, while following are examples of specifications of various kinds, which, although they may be taken as typical, must be distinctly understood as only capable of general application by alteration to suit existing circumstances.

All work of any importance at all requires a fully detailed and properly worded specification, because the document will contain all those minor details which it would be practically impossible to include in the drawings. It is, however, the practice of many engineers (and practically all architects) to put very little information on the drawings, and elaborate very much on the specification. The writer, however, is very much inclined towards the use of large-scale fully-dimensioned detail drawings in all cases where the office staff is large enough, and to curtail parts of the specification. A great failure of many, when they have once got into the way of writing a specification, is to overdo it altogether, and to use an exuberance of high-sounding language, this being entirely unnecessary; all that is required is to state clearly (and very clearly too) what is meant, and leave it at that.

Another very unsatisfactory item in a specification is stating that certain work is simply to be done to the engineer's satisfaction, and to say no more about it. Such a clause is too vague and should be avoided, especially as this is an understood thing throughout the contract.

All items which concern one particular trade or branch of labour must be very carefully grouped together.

It is the practice of many to include rough sketches in the specification. The writer considers it a bad practice, because, by reason of their usual insufficiency, they become excuses for the contractor, perhaps, scamping work otherwise set out in the drawings; besides, when many copies of the specification have to be made, their reproduction entails no small amount of labour; at the same time, even when all the details are gone into in writing, unreasonable conditions must not be imposed, because contractors usually spot such, and do not adhere to them.

Another bad practice is that of giving liberty to the contractor to choose

between two or more methods of construction. Naturally, he is sure to choose that which benefits himself most. Besides, such conditions are hardly required.

Great care must be taken to carefully compare the dimensions, levels, and scantlings on the drawings and specification, and see that they agree exactly. It is very humiliating, for the engineer, for a contractor or foreman to come to his office and point out such a discrepancy.

In a big contract, with many sheets of drawings, it is usual to make a list of these, duly numbered and referred to, and supply the contractor and others with the same. The datum on all sections should be specially noted regarding the points to which it is referred.

The other points in a general specification are as follows:—

Water, stating whence it is to be obtained and who is to be paid for it. To lay on water is usually the first thing to do.

Scaffolding, which is supplied as specified, or as the engineer shall direct.

Fencing, which must in all respects be an adequate safeguard against trespass, damage to property, and straying cattle.

Covering and protecting all works in frosty weather, and making good damage resulting by reason of neglect of this condition.

Watching and lighting at night, and during times work is not in progress.

Pumping of excessive water, and keeping the works and trenches dry.

Providing for proper sanitary accommodation for the men, and compelling them to make use of such, and the removal and finally cleansing of latrines and site after completion of the contract.

Keeping clean, and clear of all unnecessary refuse, the works while in progress.

Affording facility for thorough inspection of any part of the works, stores, tools, machinery, etc., not only to the engineer, but to his assistants and representatives. This is a clause which young engineers must take care not to abuse: these people often, by reason both of their good intentions and inexperience, likewise are liable to give the contractors unnecessary trouble by making unreasonable demands, while the use of a little judgment, without undue familiarity, will gain valuable experience at the contractors' hands.

The keeping off the works of all unauthorised persons, without reserve.

The supply of local materials and employment of local labour. The term "local" to be decided finally by the engineer.

The engineer's decision to be in all cases final and binding.

Proper office accommodation to be provided for the resident engineer, with fuel and light.

Proper measuring-boxes to be provided for use on the works.

Inspection before use of all old material to be used over again.

Inspection of foundations when open. Samples of all materials to be submitted before use to the engineer's office or other place which he shall determine, and the right to inspect any material at any time, and if necessary to condemn the same and order it off the works at once.

(This should be rigidly insisted upon.)

Proper hoardings to be provided, and customary application to be made to the local sanitary authority. The right of advertising on same to be stated, or its prohibition enforced.

Making good all pavements, street channels, kerbstones, etc., and providing proper footway for pedestrians, and accepting liability for any accidents or claims for injury or damage.

Each delivery of pipes to be accompanied by an invoice, which is to be delivered or posted as may be directed by the engineer.

Contract may be dissolved.—Should it at any time appear to the engineer that the contractor is not complying with the terms of this specification, or not delivering the pipes in the manner required, the corporation shall have it in their power to dissolve the contract, and to agree with any other party or parties for the completion of the work, the original contractor remaining liable to the corporation for any extra price they may have to give, or for any extra expense, loss, or damage which they may sustain in consequence of his default, and any balance or sum of money which may be due to him at the time of dissolving the contract shall be retained by the corporation, to be applied in or towards the payment for any loss or increased expense.

Contract.—The contractor shall enter into a formal contract with the corporation when called upon to do so, and must give satisfactory security for the due performance thereof, which contract shall contain all the usual stipulations.

The corporation do not bind themselves to accept the lowest or any tender.

Shipping—Pipes broken during shipment at the spigot end for not more than one quarter of their length will be accepted at a proportionate price, the contractor bearing the expense of cutting off the damaged portion.

Note.—Pipes are usually shipped without packing, but to decrease the risk of breakage either straw ropes or W.I. hoops are strongly recommended to be used to protect the spigot ends of the pipes.

If W.I. hoops are wanted, same are shrunk on while hot.

If straw rope, they must be securely fastened on by wire.

SPECIFICATION NO. 2. FOR LAND DRAINAGE.

The minor drains are to be cut in parallel straight lines directly up the natural slope of the land, or otherwise in such direction and at such distances apart as may from time to time be pointed out, to be ft. deep at the distances apart, generally of ft. as shall be directed. The drains are to be cut as narrow at the bottom as practicable, allowing space for the thickness of the pipe. The sides must be dressed clean, and the bottoms have clean and uniform inclinations. Extra cutting must be given through rising ground or irregularities of the surface.

The main drains are in all cases to be 6 in. deeper than the minor drains.

None of the drains to be filled till inspected by Mr , who will superintend the work, and all the material, excepting stones, is to be returned to the drains as equally as possible.

In soft clay or peaty ground, where a firm bottom cannot be obtained, precautions must be used by cutting an extra depth and laying turf with the green side up in the bottom, or boarding, upon which the pipes are to be placed, or such other method as may be directed.

The work is to be carried on by first cutting or preparing a thorough outfall to the main drains. The outfall to be a foot below the mouth of the main drains. All the drains must be filled, keeping the bottom of the drains clear of mud or loose soil as the work proceeds. The contractor is to perform all boring and blasting of stones, and pay the cost of the whole work excepting the pipes, which are to be furnished by the employer, laid down as conveniently as may be, and excepting also the laying of the pipes, which is to be done by

a man appointed and paid by the employer. The work must be completed to the satisfaction of Mr , or any competent party the employer may appoint to inspect the same. Estimates to be given in per imperial chain of 22 yds. -in. cylindrical pipes to be used in the minor drains, and -in. main pipes, and the work to be finished by .

SPECIFICATION NO. 3. FOR A MAIN SEWER.

2. *Excavation, etc.*—The contractor to excavate the trench to the depths and width required and to the levels shown on the drawings, and to guard, watch, and light the excavations or other works and materials that may be dangerous to person or property. He must also preserve the surface metalling and hard material, to be afterwards replaced towards restoring the road surface, making good deficiencies with broken metal, for which purpose he must supply broken whinstone to pass through a 1½ ring, and spread and roll the same; where the cutting passes through fields, the sods must be carefully taken off, rolled up, and replaced after the trench has been filled in, and properly rammed down.

3. *Rock cutting.*—Should any rock occur in the excavations, the same is to be cut away or quarried by the contractor at his own cost, without any extra charge.

4. *Pipes, and laying and jointing.*—The pipes to be of the best quality Scotch fireclay without fire cracks or other imperfections, extra salt-glazed inside and out, of true bore throughout. The pipes to be laid in perfectly straight lines between the manholes, and to the gradients shown on the sections, on a bed of Portland cement, gauged 6 to 1 and 18 in. x 6 in.* Pegs to be driven in the bottom of the trenches to indicate the top surface of the bed of concrete, which is to be accurately boned out throughout the whole length to the required gradient, with proper posts and sight rails; and when it has set, the pipes are to be laid upon it and jointed. The joints to be made with tarred gaskin and pure Medina cement, and the pipes well cradled to half their height with concrete 6 to 1.† The Medina cement should only be mixed in quantities enough to joint three pipes at a time, and no water to be added after once mixed. When the jointing has been completed from manhole to manhole, the pipes are to be filled with water and tested by a head of at least 2 ft. over the soffit of the higher end of each length of sewer respectively, and must stand the test to the complete satisfaction of the engineer; the cradling then to be done, all earthy or other matters to be removed from the pipes. Place 6-in. junctions at various places directed by the engineer, and those not required at present to be carefully stopped with slate and lime mortar.†

5. *Filling the trenches.*—When the sewers have been tested, inspected, and approved of by the engineer, the trenches are to be filled with earth in layers of not more than 12 in. in depth and properly rammed. The roadway to be restored to its original position, as specified in No. 2 paragraph.

6. *Manholes.*—Construct all manholes in the shape of the frustum of a cone, 5 ft. in diameter at the bottom and 2 ft. at the top, of concrete, gauged 6 to 1. Walls to be 15 in. thick, rendered in cement on the inside and floated to a smooth hard surface, and laid upon a foundation of concrete 1 to 6, 12 in. deep under invert of channel. The concrete to be banked up on each side to a height of 9 in. over the invert, and sloped off and floated to a hard, smooth

* For a 12-in. pipe. Other sizes in proportion.

† Special stoppered junctions are advised.

surface in pure cement. The manhole covers to be the best pattern heavy cast iron, 19 in. in diameter, with cast-iron frames. Ham Baker's No. 176, P.C. 23, to be used throughout, except the engineer should direct otherwise. Strong wrought-iron steps to be supplied to each manhole. Paving with hard granite setts to be supplied round the covers, bedded in Portland cement concrete 6 to 1, and 6 in. deep, and pointed with Portland cement mortar 1 to 3.

7. *Lamp-holes.*—Construct lamp-holes at midway distance between each 2 manholes, with 9-in. fireclay pipes encased with Portland cement concrete 6 to 1, and 6 in. thick, flushed up on top to receive cast-iron frame and cover 10 in. in diameter. Ham Baker's No. 175 to be used throughout, the 12-in. pipes to have 9-in. square junctions to receive the lamp-hole pipes.

8. *Gullies.*—All gullies to be connected by a 6-in. fire-clay pipe, extra salt-glazed, and laid on a bed of Portland cement concrete 12 in. x 4 in., and jointed, etc. similar to main sewer. If any gullies should enter a manhole it must be at a level of 6 in. over the invert, with a 6-in. half-round channel. Those entering the sewer must be in a straight line from the gully to the sewer junction, at an angle of 45 degrees.

9. *Cement.*—The cement to be of a brand and quality determined and approved by the engineer.

10. *The Works.*—The whole of the works to be executed in the best and most approved manner, with materials of the best quality, to the entire satisfaction of the engineer.

11. *Fine and Penalty for Delay.*—The entire works under the contract to be completed in months from the date of signing the agreement, under a penalty of 10s. per day for every day afterwards until the date of final completion.

12. *Maintenance of Work.*—The contractor shall maintain and uphold the entire work for a period of three months from the date of completion of the work, keeping the roadway along the sewer well flushed up with clean gravel. Any obstructions to be removed, and the sewer kept in good working order throughout.

13. *Extras.*—No allowance shall be made to the contractor for any extra work without the written order of the engineer.

14. *Alterations.*—The sanitary authority reserves the right of making additions to or deductions from the work without violating the contract; such additions or deductions, if any, will be valued at the prices in the estimate accompanying the contractor's tender.

15. *Responsibility of Contractor.*—If the contractor or his workmen break or injure water pipes, gas pipes, or any other property whatever, he must make good the same at his own cost. He will also be held responsible for any accident to person or property owing to want of care or skill on the part of himself or his workmen.

16. *Dispute.*—If any dispute arise upon the quality of materials or workmanship, or mode of execution of work, it is to be understood that the decision of the engineer is final and binding.

SPECIFICATION NO. 4. FOR A ROADWAY.

Quality of Materials.—The quality of work, method of procedure, and material are all to be of the best of their various kinds, and to be as set forth in the conditions of contract and specification, samples of the various materials to be submitted, and all materials used to be up to standard of same.

Site.—The site is about 1 mile from station. Due allowance for any cartage necessary must be made by the contractor and included in his estimate, and no additional claim for carting on this account will be entertained.

Dimensions and Levels.—The contractor is to visit the site, take all necessary dimensions, be thoroughly acquainted with the various falls of the ground, nature of the soil and subsoil, and features of the site, and he will be held responsible for the correct dimensions of all setting out for slopes, excavations, and embankments required, and must duly allow for same in his contract. No claim for additional work or labour due to alleged incorrect information or dimensions in specifications or drawings will be considered, acknowledged, or entertained in any way whatsoever.

Water.—The contractor is at his own cost to provide a due and necessary supply of water for the efficient carrying out of the works comprised in the contract, and is to pay all fees and charges for the same.

Details.—Detail drawings will be furnished as the works proceed where required in the specifications; these are to be considered as embodied in the contract, and no claim for additional work alleged to be due by reason of same will be considered.

6. The contractor is to provide, erect, and maintain all necessary fencing for enclosing the land, and will be held responsible during the whole of the time the works are proceeding for any damage to the surrounding fields that may occur by reason of trespass of workmen employed on the work, or by reason of cattle or persons straying on to the site of the works, and he shall not allow the site of the works to be used or serve as a public footpath or thoroughfare before the entire road is completed and handed over to the local authorities. He shall construct between the road and parts of the land cut off by the same from the adjoining pastures indicated on the plan a substantial fence; such fencing to be either stout hurdles pegged together, sunk into the ground, and with a continuous head lath along the top of same, or larch posts 3 ft. 6 in. out of the ground and 10 ft. apart, with 4 rows of galvanised iron strand wire. The bottom ends of the posts to be tarred before being sunk into the ground with best gas tar, and the ends of the wire to be twisted twice round the poles, and secured by galvanised iron cleats, and wires to be cleated to posts with galvanised iron cleats.

The works to be closed during the whole time of the contract, and the contractor is to be held responsible for any trespass resulting from persons using the road as a footway or highway until the date of payment of the retention money.

Any cement used to be of a quality and brand approved of by the engineer, and be subject to tests as stated.

7. *Granite Lintels.*—The manholes where shown to have granite lintels of not less width than 8 in. each, to be set and fitted tight, and jointed and headed in cement mortar, and to extend on all sides the full width of the brickwork.

8. *Hard core.*—Hard core for the road and footway. Foundation to be broken brick or other approved material of good quality, clean, and free from dirt, earth, clay, mud, and old mortar. Sample to be submitted to the engineer and approved by him before any material is placed upon the ground.

9. *Stone.*—The stone used to be broken granite, green whinstone, or other approved stone, and free from dust, earth, or dirt, and all stones to be broken and pass through a 2-in. ring.

10. *Gravel*.—The gravel for the footpaths to be clean, fine, well-washed $\frac{1}{2}$ -in. gravel of approved quality, and to be well washed and rolled.

11. *Fine Gravel*.—The fine gravel for binding material to be clean road scrapings or other approved material, free from all foul and objectionable matters.

Road-making.

The roadway is to be constructed as follows:—The surface is to be taken away, and the turfs neatly stacked in a convenient position, and to be used for covering the slopes; the ground is to be then excavated to the depths and to the contour on section for the formation level, and the road surface well rolled with a steam roller to the correct contour, until the whole is hard and firm in the opinion of the engineer; on this is to be placed not less than 9 in. of hard core, and the whole well watered and rolled with a steam roller to the correct contour; on this is to be spread not less than 6 in. of broken granite, to be well watered and rolled, and clean approved binding material swept over until the whole is firm and solid, and finished throughout to an even contour. The road, after rolling hard, to show 6 in. and 9 in. thick of granite and hard core respectively.

13. *Footpath*.—The footpath to be excavated to a depth of 9 in., not less than 6 in. of hard core filled in, watered and rolled and covered with not less than 3 in. of clean, fine gravel. The path to be formed exactly as the road way, but hand rolled instead of steam rolled, and to be finished to the full dimensions specified.

14. The contractor to have upon the works a steam roller of 10 tons weight, and to use the same as directed, finding all coal, water, wages, and expenses therewith.

15. The granite kerbs to be 9 in. $\times$ 4 in. set on edge, neatly squared on two faces, to be in lengths of not less than 2 ft. 6 in. each, and to hold up to full dimensions throughout, to be well bedded in concrete, and to show a face to channel of 5 in. in depth. Sweeps to be allowed for where shown on plans, and to be set firmly and struck to true radius. The heading joints to be perfectly square.

16. *Channels*.—The channels to be formed for a width of 9 in. in granite as the road surface, but all to be large stones, all well bedded and packed.

SPECIFICATION NO. 5. FOR STEAM ROLLING.

1. The work will consist of in the aggregate about hours of effective rolling on such roads in the district and on such parts of them as the surveyor may specify.

2. The work is to be commenced within seven days after the contract has been signed, and carried on with all reasonable despatch until it has been completed.

3. For the purpose of the work, the contractor is to provide, properly equip with necessary tools and appliances, and maintain in good working order until the work is completed, one or more steam rollers, each weighing $12\frac{1}{2}$ tons empty, and of a make and design to be approved of by the surveyor.

4. For each roller he must provide a steady, qualified foreman driver and careful flagman.

The latter must be provided with brushes, and assist in sweeping when the exigencies of the traffic permit, and when called on by the urban district overseer.

6. Four notice-boards, "Beware of the Steam Roller," printed distinctly in white letters 4 in. in height on a black ground, are to be provided with the roller, and fixed before the roller starts work each day, at the side of the road, at a short distance from the limit of the travel of the roller, each end.

7. The contractor is to take every needful precaution for the safety of the public during the progress of the work, as he will be held responsible for the results of any accident which may occur through neglect on the part of his men, or through failure of or defects of his plant.

8. The surveyor may order the dismissal of any man employed by the contractor without assigning any reason for so doing, and a fine of £1 will be imposed for each day such order is disregarded.

9. The surveyor may also order any implements or stores which may be wanting, and which he may consider necessary for the effective working of the plant, to be supplied at the expense of the contractor.

10. Only such times as the roller may be actually at work will be allowed in reckoning the number of effective hours on which this contract is based.

11. No allowance will be made for time lost during repairs, or by reason of the absence of any of the contractor's staff, but for such time as the roller may be unable to work through lack of material for rolling, such allowance will be made as the surveyor may consider fair.

12. The surveyor may reduce the number of hours of rolling to any total not less than half that above specified.

13. Should the total number of hours worked be less than that above specified, the amount of contract will be reduced by a sum equivalent to the value of the time not worked, at the rate of 2s. 8d. per hour (price, of course, varying).

14. The total number of cubic yds. of broken stone rolled in and finished off in any one week must not be less than 225 in thorough sheeting and 180 in patching.

15. If in any case a less quantity than that above specified is rolled in, a deduction at the rate of 1s. 2d. per ton in the one case and 1s. 6d. per ton in the other will be made, unless reasons which the surveyor may consider satisfactory are given.

16. The total cost of rolling, exclusive of the cost of the material and labour supplied by the road contractor or surveyor, is not to exceed 1s. 6d. per cubic yd. of the material rolled in.

17. The contractor must, when so directed, re-roll at his own expense any portion of a road which the surveyor may consider imperfectly done, or which may show signs of disintegration within a period of one month after the first rolling has been completed.

18. The contractor is, if called upon to do so before the date fixed for completion of the contract, to carry out any rolling extra to that above specified which the surveyor may order to be done in the district, at the same rate and on the same conditions as those laid down herein for the contract proper.

19. The work is to be carried out under the supervision of the surveyor, his assistant and overseers, whose orders the contractor must carry out, and completed to the entire satisfaction of the surveyor.

20. Rolling is to be discontinued during frost, or at any time the surveyor may so direct, and no allowance will be made for any loss the contractor may suffer thereby, unless as provided in clause 13.

21. Payment to be made on account as the work proceeds, on the certificate of the surveyor.

22. With each roller the contractor is to provide and maintain in good order a water cart of approved design and capacity, with proper hand-pump suction-hose stand-pipe if required, and all necessary fittings.

23. The contractor is to pay to any persons named by the surveyor such a sum or sums for work or materials as the surveyor may direct, and is, when called on by the surveyor, to produce the receipt for such payments.

24. For all sums so paid, credit will be given by a reduction of the specified hours of rolling calculated at the rate of 2s. 8d. per ton.

SPECIFICATION NO. 6. CONCRETE, AND CONCRETE STEELWORK.

Cement.—The whole of the cement used on the works shall be of the best highly-burnt pure Portland cement, of a uniform dark-grey colour, and free from lime, slag, dust, or foreign material after leaving the kiln; it shall in all cases not weigh less than 112 lbs. to the imperial (stricken) bushel when poured in quietly, and purchased from some approved manufacturer. The cement shall be ground extremely fine, and so that 98 per cent. at least will pass through a sieve of 5776 meshes to the sq. in. The cement, when gauged neat, shall be capable of sustaining a tensile stress of 400 lbs. per sq. in. of sectional area eight days after having been made in a mould, and immersed in water during an interval of seven days. At the expiration of twenty-eight days the briquettes shall have shown a steady increase by bearing a tensile stress of 580 lbs. per sq. in., having been immersed in water during the last twenty-seven days. The cement will further be tested along with the sand to be used in the work. It will be gauged with three times its weight of clean, dry, sharp sand, which has passed through a sieve of 400 and been retained on one of 900 meshes to the sq. in.

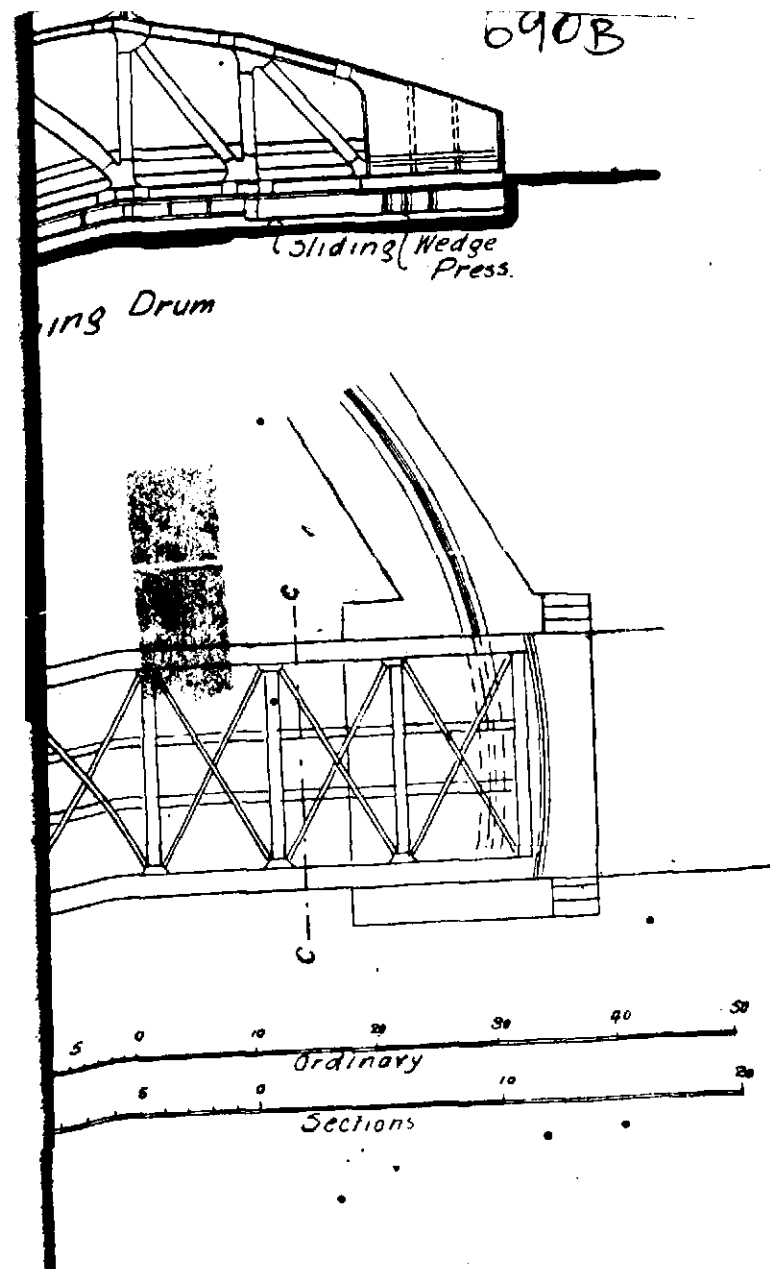
The cement and sand having been well mixed dry, about 10 per cent. of their weight of water is to be added, and the briquettes formed in the mould. The briquettes, which in the meantime have been kept in a damp atmosphere, are to be put in water twenty-four hours after having been made, and will remain in water for a period of twenty-eight days from the time of gauging, when they will be required to stand a tensile strain of not less than 180 lbs. to the sq. in.

In addition to these tests, the cement shall, when subjected to a chemical analysis, yield a satisfactory result, and shall have a specific gravity of 3.1.

The contractor shall provide, at some central place in the vicinity of the works, a suitable shed or other approved building, having a damp-proof floor, with all necessary subdivisions so that each consignment may be kept separate, of ordinary size, to contain such quantity as the engineer may order to be kept in store.

All cement before being used shall be spread upon this floor in a layer not exceeding 12 in. in thickness, and allowed to remain at least fourteen days, or for a longer period if the condition of the cement require it, the contents of the various bags and casks being thoroughly mixed with each other in spreading out on the floor, and the cement shall be turned over from time to time as required by the engineer.

The contractor shall take precautions to keep all cement dry, and shall provide all necessary waterproof sheeting for the protection of the same on the works.



The contractor shall always keep sufficient stock for fourteen days' requirements at least, and with each delivery of cement he shall furnish the engineer in writing the quantity sent in, the maker's name, date when taken from kiln, and samples taken from the consignment, in the presence of the clerk of works, for testing purposes, and deliver the same at the properly marked and labelled.

The cement will be tested by the engineer, and all that fails to comply with the above standard of strength will be rejected, and must forthwith be removed at the expense of the contractor, and an average of six briquettes shall be considered conclusive. No cement to be used until after being tested.

The first consignment of cement shall be put in the store one month before the commencement of the work for the purpose of being tested by the engineer, in accordance with the terms of this specification, and the engineer and clerk of works shall have free access to the store at all times, and shall be at liberty to take samples of cement when they may desire.

Gauge-boxes.—The contractor shall provide suitable gauge-boxes for measuring the proper proportion of cement, sand, or ballast to be used. All material shall be measured in these boxes as the work proceeds.

Sand.—Only clean sand shall be used. It shall be coarse and sharp, with grains of various sizes; and also contain the smallest possible percentage of voids; should the sand contain more than 5 per cent. of loam it will not be allowed.

Aggregate.—The aggregate shall be composed of broken stone or gravel. The broken stone should be of a hard, close-grained quality, and crushed so that its largest dimension shall pass through a ring 1 in. in diameter. Gravel should be free from dirt, and in size ranging from that of a pea to 1 in. diameter.

Proportions.—All concrete for slabs, beams, and columns shall be proportioned, one part of Portland cement, one and a half parts of sand, and two and a half parts of broken stone or gravel.

Mixing.—The concrete shall be (preferably) machine-mixed, using either a batch or continuous mixer of approved design. Plenty of clean water should be used, so that the resultant mixture may be known as a wet mixture. All materials must be first thoroughly mixed dry, after which the proper amount of water is added, and the mixing should continue till the concrete is uniform. A competent foreman must be in constant attendance at the mixer, and give his approval to every batch which leaves the machine.

Reinforcing Steel.—No reinforcing steel will be considered which does not provide for shearing stresses as well as direct tension. These shear-resisting members should be preferably inclined at 45°, pointing up and towards the supports of the structure. The shear members must be rigidly attached to the tension members. Sufficient steel must be placed so that the concrete will be obliged to resist only direct compression and shearing stresses up to 50 lbs. per sq. in. No steel shall have at any point less than 1 in. of concrete covering. Steel bars shall not be painted. A slight film of rust will not be objectionable on the same. In no case will steel of a higher elastic limit than 45,000 lbs. be considered. The same shall have a tensile strength of from 60,000 to 70,000 lbs. per sq. in., with an elongation of not less than 20 per cent. in 8 in. A bar should bend when cold around its own diameter through 180°, and close down upon itself without cracking.

Placing Concrete.—Concrete shall be placed as rapidly as possible after mixing, and shall be thoroughly puddled immediately thereafter.

When concreting is once commenced it should be carried on rigorously to completion if possible. If concreting must be stopped before an entire floor is completed, the stop shall be made in the centre of the beams and centre of floor slabs. The place where the concrete work is stopped must be at right angles to the direction of the beam or slab. In no event shall work be terminated in beams or floor slabs where the future shearing action becomes great, as at their ends, or directly under a concentrated load.

Frosty Weather.—In frosty weather special precautions must be taken to protect the exposed surfaces. Concrete shall be placed in frosty weather only when the same cannot possibly be prevented, and then especial precautions must be used to cover the work at once with at least 5 or 6 in. of sawdust

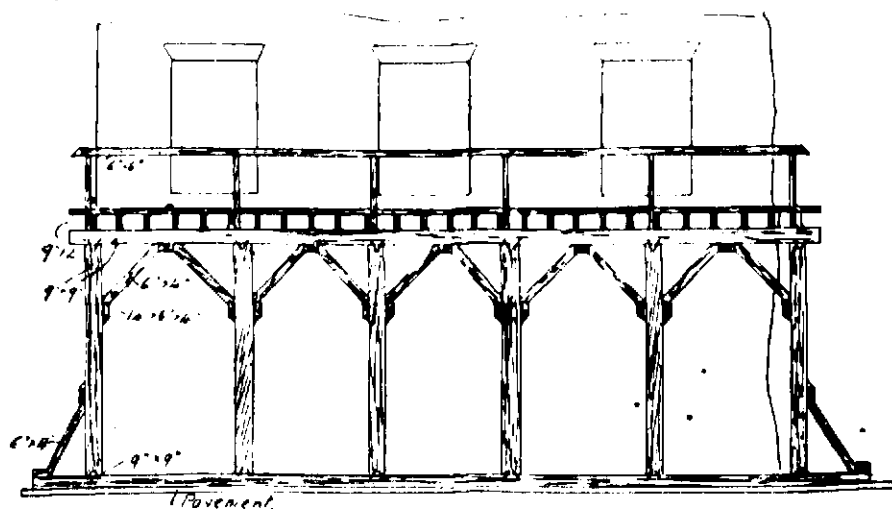


FIG. 1020.

or manure. The forms for such work must be left in place at least three weeks longer than customary, and under no circumstances shall they be removed unless the concrete has thoroughly set and the frost entirely disappeared. (It is by all means preferable to avoid concrete work in frosty weather.)

Centering.—The centering must be true, rigid, and properly braced, and of sufficient strength to carry the dead weight of the construction as a liquid without deflection. All joints should be fairly tight, to prevent leakage of liquid masses. The design of the centering to be such that the sides of the beams can be taken down first, then the slab centreing complete. Centering should never be removed in less than four weeks' time after concreting, or until the concrete has thoroughly set, and has aged to give it sufficient strength to carry its own weight besides any live load that is liable to come on the work during the course of construction. In no event shall the false work be removed until approval is given by the architect or clerk of works. Beams shall remain supported for at least two weeks after all other false

work has been removed. Columns shall not be given their full loading in less than five weeks after concreting. After the forms are removed any small cavities or openings in the concrete shall be neatly filled with mortar if necessary. In order to obtain a smooth finish, wash with a thin grout of mortar of the consistency of whitewash, mixed in the proportion of one part of cement to three parts of sand. The wash to be applied with a brush.

Tests.—Floors shall be tested after the centering has been removed two months to a uniformly distributed load equal to twice the safe live load; with this load there shall not be a deflection exceeding $\frac{1}{320}$ part of the span, and the floor should return to its normal position after removal of the load.

Finished Surfaces.—The cement finish for floors shall be proportioned not less than 1-2, using in all cases specially sharp, clean, and gritty sand. It shall be trowelled to a thoroughly smooth and even surface. Cement finish, when applied to a concrete base, must be laid at the same time as the base, and shall not be less than $\frac{1}{4}$ in. in thickness. If laid after the base has been finished, the cement finish shall be at least $1\frac{1}{2}$ in. thick.

In this chapter mention has been made of gantries. This term here has special reference to what are known as platform gantries, and is used on confined building sites where space is limited, so as to keep a clear pavement. In large cities their use is essential, both to the satisfactorily carrying out of the work and the safety of the public. A gantry is erected in the following manner: The sleepers are first laid down, the ground being properly levelled up to receive them. The standards (of the same section as the sleepers) are then hoisted into their correct positions and securely "dogged" to the sleepers on both sides; on these the heads are laid, also of the same size as the sleepers, and "dogged" to the standards. The cleats are now spiked to the standards, sleepers, and heads about 6 ft. 6 in.

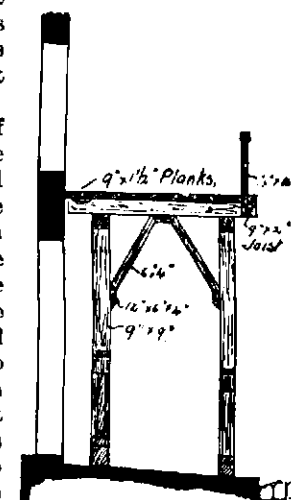


FIG. 1021.

above pavement-level, so as to keep the struts up out of the way. Then the struts are fixed in position; these are kept as near as possible to an angle of 45° to the horizontal. Where the struts meet the head a short "straining-piece" is placed between them, spiked to the head. This is made longer or shorter so as to keep the struts to the angle of 45° . Over the heads are laid the joists at from 12-in. to 24-in. centres, according to the load the gantry will be expected to carry, and on these are laid either $1\frac{1}{2}$ -in. scaffold boards or 2-in. or 3-in. planks, as requirements call for. A handrail and guard-board should be placed along the front, so as to prevent anything falling over into the roadway.

The usual scantlings for this class of work are (a drawing appears in figs. 1020 and 1021)—

Sleepers, standards, and heads, 6 in. by 6 in. to 12 in. by 12 in.
 Struts and stays, 4 in. by 3 in. to 8 in. by 8 in.
 Cleats, 1 ft. by 4 in. by 3 in. to 1 ft. 6 in. by 8 in. by 8 in.
 Top joists, 7 in. by 2 in. to 9 in. by 3 in.; centres, 12 in. to 24 in.
 Planks, 9 in. by $1\frac{1}{2}$ in. to 9 in. by 3 in.

In contractors' yards and yards used for heavy stores in connection with a city engineer's department we have a traveller gantry very often, which is somewhat similar in construction, but without the top joists, planks, and rails. On the top face of the heads are laid common railway rails, on which the traveller runs. It is obvious that the space between the two gantry frames must be free from obstructions, so as to provide a free passage for materials suspended from the traveller. Therefore all struts and stays to provide lateral support must be on the outside of the frames, and be anchored to the ground by means of lengths of stout timber set in blocks of concrete.

CHAPTER XL.

ELECTRIC TRAMWAYS.

THE subject of electric traction is a large one,—so large, in fact, that a very considerable body of professional men have made and are making it a life study. It is not, therefore, our intention to treat the matter here at any length, but to leave electrical equipment, power stations, tram service, etc., to those who are specialists in this class of work, and to confine our present remarks mainly to the permanent way, and to a cursory glance at distribution systems.

Rails.—Tram rails must fulfil the following conditions:—

1. They must in no way interfere with the free passage of ordinary vehicular traffic.
2. The width of groove, while fulfilling condition 1, must not be so small as to give constant annoyance by being a receptacle for dirt, incapable of easy removal.
3. Strength and rigidity are essential.
4. Proper bonding is also essential.
5. Rails must resist wear well, because renewal is an expensive item.

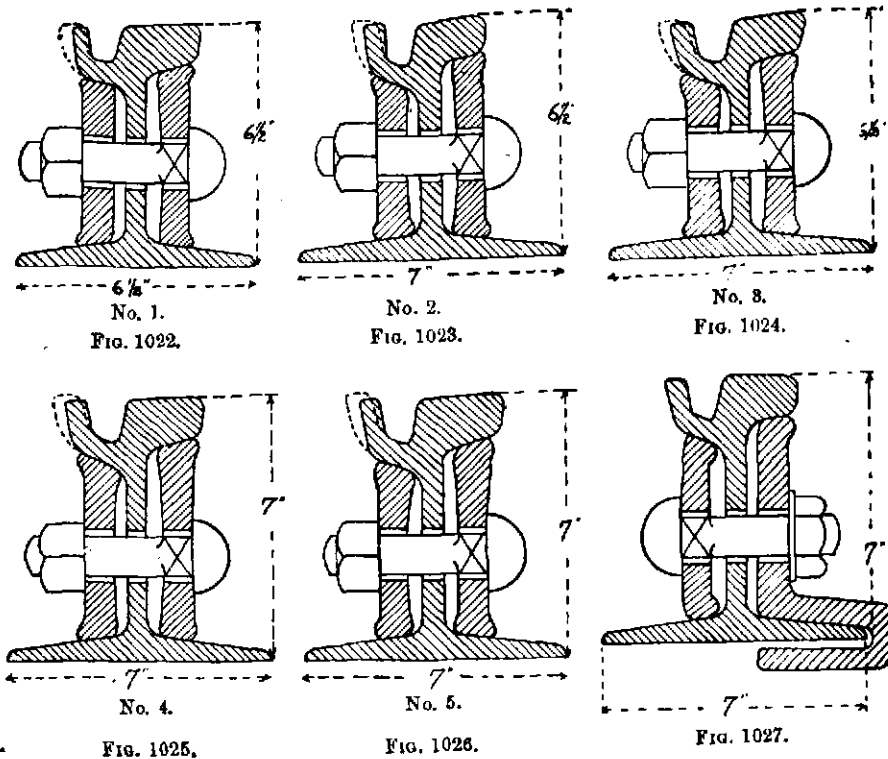
These conditions have given rise to the form of rail known as the girder rail, which is now practically universally used in the United Kingdom, but in America various types of rail, having a very wide groove, are used. The most common types of girder tram-rail used in modern tramway practice are shown in figs. 1022 to 1027, with their respective weights in lbs. per yard shown in table. As before stated, the actual sectional area may be approximately found by dividing this weight by 10. Modern practice tends to using very heavy rails, over 100 lbs. per yard. It is also useful to know that if we multiply the weight in lbs. per yard by 1·571, we shall have approximately the number of tons of rail used per mile of single track; and by ·977, the same per kilometre.

The following are the rails recommended by the British Engineering Standards Committee—(next page).

The sections C are specially recommended for use on curves. Full specifications are issued by the Engineering Standards Committee, and will not be repeated here, but the usual conditions and tests are as follows:—

In chemical composition, samples must not exceed a greater percentage of carbon than ·55, nor less than ·40; of manganese, 1 per cent. and ·70 per cent.; while the silicon must not exceed ·10 per cent., phosphorus ·08 per cent., and sulphur ·08 per cent.

The rails are usually tested as follows, samples being chosen by the inspector at random. A piece of rail is placed on bearings over a span of 3 ft.

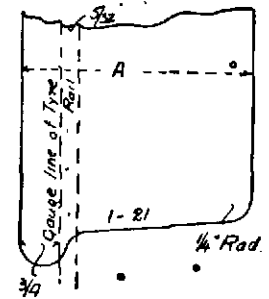


The rails illustrated here are those recommended by the British Standards Committee. The numbers 1, 2, 3, 4, 5 refer to the numbers which these rails bear in the committee's list. The dotted lines show the enlarged lip and groove which rails for special use on curves have, and the sections so designated are known as 1C, 2C, 3C, 4C, and 5C. The bolts are 3/4 in. diameter, and have square necks as shown; while the holes in the fish-plates on this side are 1 1/2 in. sq., while those on the other side, where the nuts are, are 1 1/2 in. round. A table of the rails is given below. The holes in the web are 1 1/2 in. round.

The fish-plates are 24 in. long. For sections 1 and 1C, 50 lbs. per pair; 2 and 2C, 49 1/2 lbs.; 3 and 3C, 49; and 4 and 4C, and 5 and 5C, 56 1/2 lbs. per pair. Square nuts are used, and snap heads to the bolts, and are in accordance with the Whitworth standard as regards the thread.

Section No.	Depth of Rail.	Width of Flange.	Weight. lbs. per yd.
1	6 1/2 in.	6 1/2 in.	90
1C	6 1/2 "	6 1/2 "	98
2	6 1/2 "	7 "	95
2C	6 1/2 "	7 "	101
3	6 1/2 "	7 "	100
3C	6 1/2 "	7 "	106
4	7 "	7 "	105
4C	7 "	7 "	111
5	7 "	7 "	110
5C	7 "	7 "	116

6 in., and a ton weight is allowed to fall on the rail from a distance of 15 ft., the actual length of the test-piece being 5 ft. for rails under 100 lbs. per yd. For rails over 100 lbs. per yd., 18 ft. is the required distance, while the steel used must have a minimum tensile strength of 40 tons per sq. in. with a maximum elongation of 12 per cent. in 2 ins. Such rails on straight lines should bear a traffic of about 100 millions of tons, while greater results may be easily expected in many cases. The use of iron rails has, of course, quite gone out of vogue; their life was comparatively short compared with modern steel rails, because, when the signs of wear appeared, it was in the form of flaking and splitting off in small pieces at the edges, while in a steel rail quite purely legitimate wear down of the section takes place, so that the rails could, if practicable, be used till the head had nearly disappeared. As it is, a good steel rail may be reckoned to lose about 5-8 lbs. per yd. in 18 years, the depth of the abrasion being about 1/8 of an inch. The wear on gradients is, of course, much more rapid, while on sharp curves, especially if the trams have rigid wheel bases, the wear may be very rapid, by reason of a considerable widening of the groove. Of course a certain amount of what would otherwise be actual wear of the rails is taken by the wheel tyres, which may be stated at anything between 1/2 in. in 2000-4000 tram-miles, sometimes much less. The rapid wear of both rails and tyre would be decreased by the use of tyre sections properly proportioned to the rail, for which purpose standard sections have been drawn up, as shown in fig. 1028, the value of A being as follows:—



Section.

1 and 1C	A = 2 1/8 in.
2 and 2C	A = 2 1/8 "
3 and 3C	A = 3 "
4 and 4C	A = 3 1/8 "
5 and 5C	A = 3 1/2 "

All rails used should be rolled to templet, which should first be approved, or by the engineer, while any rails which are under 1/2 lb. per yd. less than the specified weight should not be accepted, or if more than 1/2 lb. over this standard the extra weight will not be paid for. The rails should also have the maker's name, date of rolling, and, if desired, the buyer's name marked thereon.

We now come to the jointing of the rails. The usual method is a fish-plate and bolts similar to those on railways, as shown in figs. 1029-1032. The bolts are made with cup heads and square necks as shown, and are made from the best-quality fibrous iron, the bars being tested by bending when cold to 45° out of the straight, and then straightening them out again, which must be done without sign of fracture. They are usually 3/4 in. diameter. They also must be made to templet and not differ from the specified diameter by more than 1/32 in.

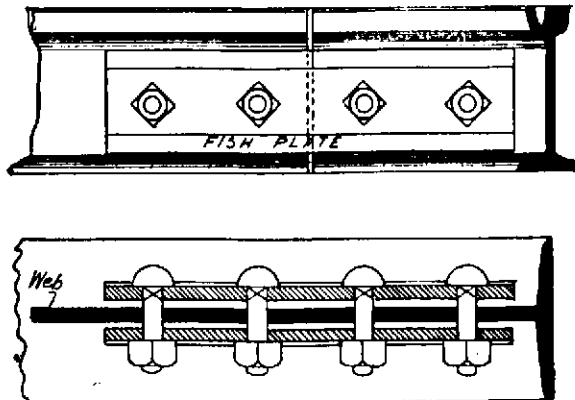
In fig. 1031 is illustrated a new patent rail-joint for tramway tracks, invented by Messrs Brown and Crosta and made by the Railway and General Engineering Company, Nottingham. The open joints on tramways always

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suffer more or less from jar of the trams passing over them, and consequently uneven wear takes place, which rapidly tends to increase it. These renewable plates are of the utmost value for remedying the evil. The writer has seen them in Dublin in practical use, where they seem to be very efficient.

The reader is doubtless aware that on railways a space (varying according to circumstances) is left between the two rails joined. This is to counteract the effects of expansion, which would distort the rails. A theory was put



Figs. 1029 and 1030.

forward, and probably rightly so, in July 1909, for the cause of the disastrous accident on the L.N.W.R. near Huddersfield, based on this fact of expansion. On tram rails, at least those which are termed "buried," that

is, set up flush with the road surface, no such expansion (at least very little) takes place; and provided the rails are laid in any but the coldest weather, this space may be neglected, and the rails butted against each other. Rails are usually made up to 60 ft. in length, and obviously the longer the rail the less labour and cost of jointing. Thus we see on an *exposed* rail, allowing the coefficient of expansion to be .0012 between 32° and 212° Fahr., that a 60-ft. rail will in-

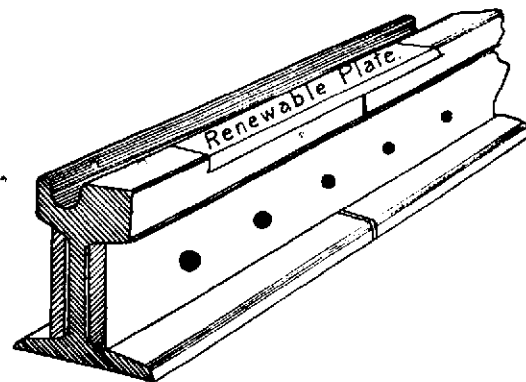


FIG. 1031.

crease in length $60 \times .0012 = .0720 \times 12 = 864$ in. = nearly $\frac{1}{8}$ in. increase in length. In hot countries this has a very important bearing.

Fishplates.—On all lines of railway, fishplate bolts will get loose by reason of the slight jar on passing over the expansion space, and to see to these fastenings a staff of special men are employed. It will be obvious to the reader, however, how impossible this is in street tramways; hence we have a

very strong reason for omitting the space referred to. Some engineers, however, still deem it necessary, for which reason the fishplate shown in fig. 1032 is devised to remedy the knocking evil so detrimental to the road-car machinery—also a strong point in favour of the 60-ft. rail. American engineers seem to disfavour the fishplate, which has led to the large adoption in that country of two forms of what are known as continuous joints: the first is the cast welded joint, which briefly consists in thoroughly cleaning the rail and pouring cast iron over the joint in a mould, producing a section like fig. 1033, while the second practical method is the electrically welded joint

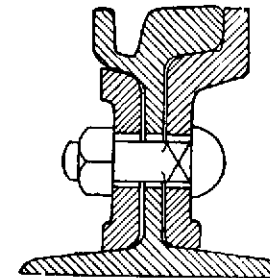


FIG. 1032.

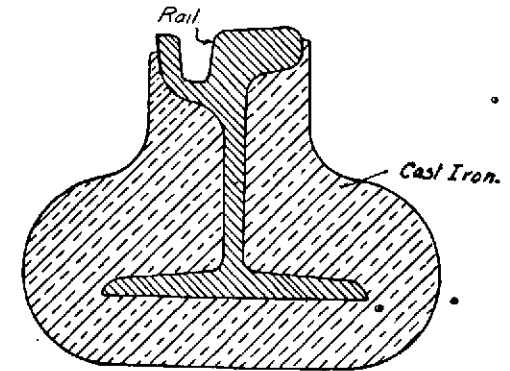


FIG. 1033.

which is shown in fig. 1034, which is made by first thoroughly cleansing the rail (upon which the whole success of the work depends) and welding the four points $\times \times \times \times$ electrically to the rail, the current being supplied from the trolley wire; a steel wedge, accurately fitted, then completes the process. In using the cast joint the common practice is to do every alternate joint over a section, giving them time to properly cool before the others are started.

Laying Rails.—The actual laying of rails on a system of overhead wires is now nearly always done by placing the rails flat on a hard bed of concrete,

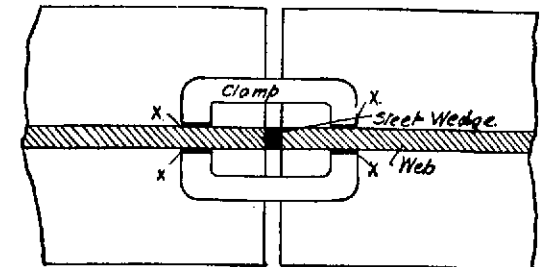


FIG. 1034.

usually about 9 in. deep. Iron tie-rods are used, and the system is shown in fig. 1035. A description of the methods of laying would take a long time, and, as a matter of fact, the proper understanding of the methods employed is best left to be gained by practical experience. A system of laying rails, not now much in vogue, is shown in fig. 1036.

Up to the present we have confined our remarks to straight and level tracks: lines on gradients and curves have certain modifications which we proceed to consider. It may first, however, be stated that for street tramways

Now the actual rail might in itself form a very good conductor, but when we get to the joint we have rust, dirt, and imperfect bond in general, which, although mechanically satisfactory, would be electrically imperfect.

Mr J. H. Rider, in his *Electric Traction*, states the resistance of a steel rail = $\frac{2.5}{\text{weight in lbs. per yard}}$ = ohms per mile, based on the fact that resistance

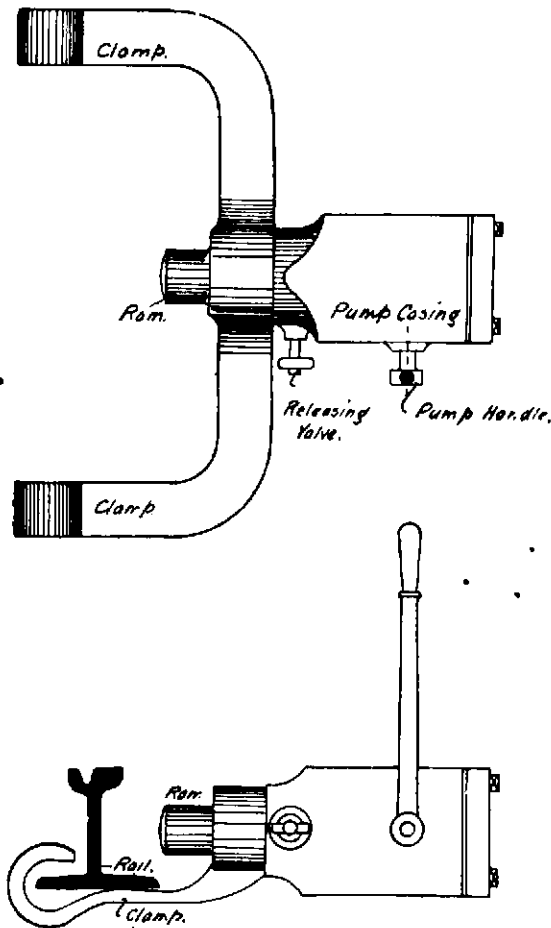


FIG. 1039.

of any conductor varies inversely with the sectional area; and as we have 2 or 4 rails, as the case may be, we have a system of conductors in parallel. On account of the high resistance of the rail joint, there will be a considerable leakage of current always seeking the easiest path back to the station. This path will probably be along water and gas mains, setting up therein rapid corrosion. The action of electrolysis in this direction is interesting to the municipal engineer. It is defined as follows:—

When a current of electricity is passed through a solution the latter is split up into its constituents, the current entering by the anode and leaving by the cathode. The constituents are called the ions. For instance, if electricity is passed through water (especially if it is slightly acid), the water is split up into hydrogen and oxygen. The increased supply of

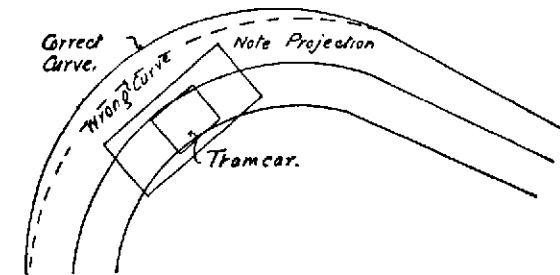


FIG. 1040.

oxygen will thus rapidly hasten corrosion of the pipes, because iron is a peculiar substance, inasmuch as nearly all elements will combine with it, and very small quantities of impurities will change its character. In fact, corrosion of iron is very often due to electro-chemical influences.

Returning to our joints, seeing that the ordinary joint is electrically

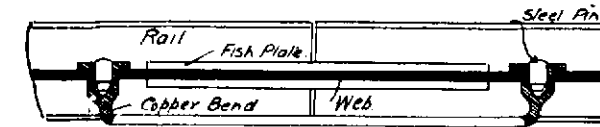


FIG. 1041.

imperfect, means have to be resorted to whereby it can be improved. The system adopted is that of bonding, which bonds, while having a high conductivity, must be flexible, firm, easily fixed, and cheap. The surface of contact with the rail must be fairly large.

One of the most successful (the Chicago bond) is shown in fig. 1041. The

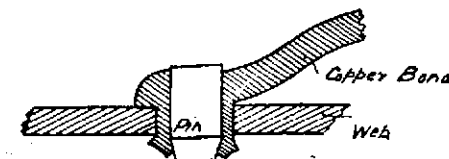


FIG. 1042.

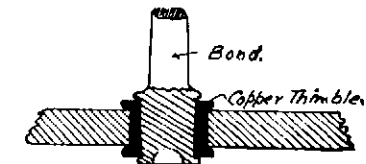


FIG. 1043.

special features are the ease of fixing by the steel pin, and the excellent bond provided thereby if the holes are made specially clean. These holes should be $2\frac{1}{2}$ to $3\frac{1}{2}$ in. clear of the fishplates. Two other bonds are shown in figs. 1042 and 1043.

Points and Crossing.—In all large towns where a rapid service of street cars is to be maintained, and where wide streets exist, double tracks are

In England 4 ft. 8½ in. gauge is invariably adopted for sake of uniformity, while the Dublin United Tramways adopt the Irish railway gauge, being 5 ft. 3 in., or strictly speaking 5 ft. 2⅞ in. Besides other considerations, the gauge has an important bearing on the comfort of the inside passengers and the capacity of the car for carrying outside passengers.

On a steep gradient there will be a continual tendency for the rails to creep down the hillside, to avoid which many engineers prefer to use what are termed soleplates. This may be either a longitudinal sleeper to which the rail is bolted, the sleeper itself being embedded in the concrete, an ordinary cast-iron chair used at the joints, also embedded in concrete, or a

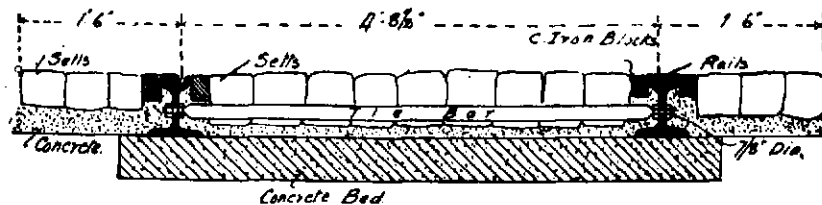


FIG. 1035.

length of old tram rail riveted or bolted to the under side of the rails over the joint: with the system shown in fig. 1036 no such anchoring would be required.

Curves must be laid to a true radius, the mean radius between the two rails always being that specified and laid down on the plans. Curves on main lines of railway want very accurately laying out and the rails properly set because of the necessarily rapid rate which the trains travel over the

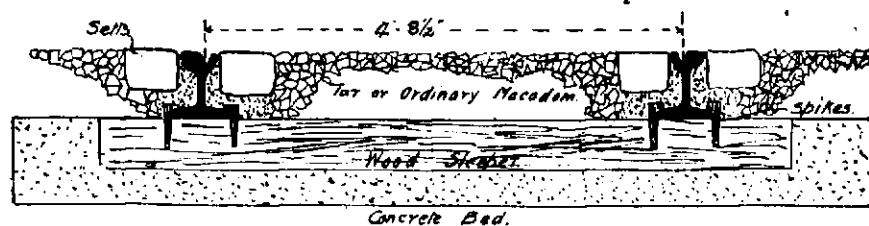


FIG. 1036.

curve. We have already briefly discussed the subject. Tramway curves are, however, much sharper, and the care and expense bestowed on railway curves is not usually to be found on ordinary tramways. With the old system of horse traction, however, the curves were rough in the extreme, simply because if the tram-car came off, the horses could pull it on again. Modern electric traction, however, demands much more attention to this part of the work than formerly, and the present practice tends towards the use of easier curves and the use of the spiral form. For the elevation of the outer rail the reader is referred to Chapter III. The practice of tapering the wheels shown in fig. 1028 tends to make the resistances on curves and straight lines nearly equal, because, while diminishing the resistance of the curve, that on the

straight increases, due to a winding movement which the cars inherit thereby. Of course, for large cars (and the modern tendency is to large ones), bogey carriages, usually having wheels of different diameters, are employed. They reduce wear on the rails and resistance of the car going round curves, at the same time being conducive to smooth running. With small cars having a rigid wheel base, which does not usually exceed 6 ft., something after the style of fig. 1037 occurs, setting up a considerable wear on the rails and consequent widening out of the groove. For bending rails a special apparatus is provided on the site. It essentially consists of a frame which grips the rail at two points on the same side about 3 ft. apart. On the opposite side, midway between these points, a great pressure is applied, either by means of a long lever and square threaded screw or a hydraulic ram. Thus the rail is acted upon by two bending moments opposite in direction, which tends to bend the rail beyond its capacity for returning to its original form. The forces set up are shown diagrammatically in fig. 1038, and a hydraulic bending machine, or "Jim Crow," as it is called, in fig. 1039.

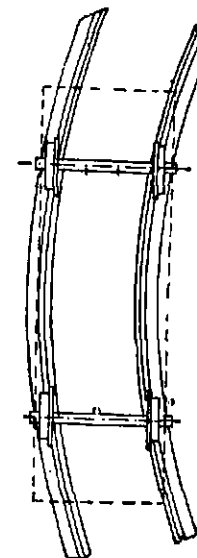


FIG. 1037.

Perhaps, on curves, the most important consideration of all is that of setting the centre lines of the two tracks further apart than on the straight. Unfortunately, sharp curves invariably occur in narrow streets, which adds to the difficulty, making it very often necessary to provide a single line only round the curve. The reason for the precaution referred to is that the car body, being rigid, tends to set itself as shown in fig. 1040; and if the extra space between the two tracks is not provided for, the first two cars that pass on that curve will collide. The necessity for this provision and the amount required is best left to the engineer to consider each case when accurate plans have been drawn up.

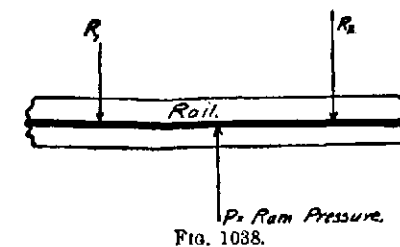


FIG. 1038.

The modern practice of tramway distribution is to use the rails as a return conductor for the current, overhead or underground "leads" being provided, the necessity for a closed circuit being obvious to those whose electrical knowledge is most elementary. In the Board of Trade regulations we see that the maximum potential difference (P.D.) between the track rails at the power station and the remote

ends must not exceed 7 volts. This demands a specially good return conductor, by providing a very low resistance in that conductor. It would be too much here to explain the various electrical terms we are using. I assume, however, that the most elementary reader understands Ohm's law, stating that—

$$\left. \begin{array}{l} \text{If } C = \text{current in amperes} \\ E = \text{P.D. (volts)} \\ R = \text{resistance (ohms)} \end{array} \right\} C = \frac{E}{R}.$$

invariably laid down. In smaller districts, however, and where narrow streets exist, single tracks only are usually provided. This necessitates two things—

1. Passing-places.
2. Means of changing over at the termini. Besides this, on a double-track system, means of occasionally allowing trams to pass from one line to the other have to be provided, and in both cases junctions will be required throughout the system.

Let us first consider passing-places on single tracks. These are usually known as "turnouts." On the old horse lines fixed points were used: a simple guiding of the horses by the driver and special mechanical construction

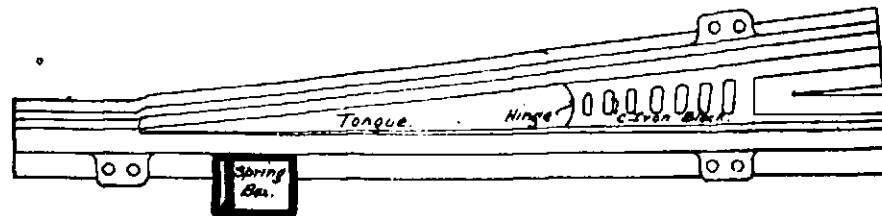


FIG. 1044.

enabled the car to be directed to the proper rail. Such contrivances are not suited to electric traction, and movable points are used. A point consists of specially formed taper rails, cut in a planing machine. The designing of such contrivances is usually left in the hands of engineers and draughtsmen accustomed to such work, the whole framework, provided it is of reasonable dimensions, being prepared and put together in the erecting shop, or those parts which are considered most convenient, the members being fixed to plates, bolted together by means of bolts and distance-pieces, and finally made firm by "blocking" with cast-iron wedges. The work demands skilful attention

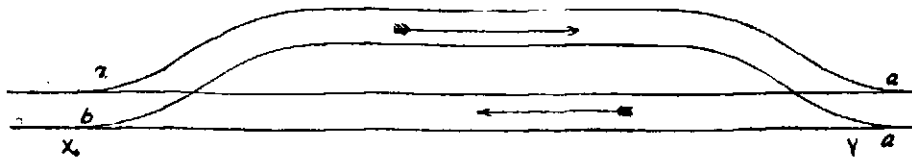


FIG. 1045.

by experienced workmen. On railways the movable tongue is moved mechanically from the signal boxes, but on tramways three systems are employed—

1. Automatic spring points.
2. Electrically-operated points.
3. A tongue, capable of hand movement by means of a light iron rod.

A spring point is shown in fig. 1044. They are invariably used for turnouts, as shown in fig. 1045.

The direction of the cars is shown by the arrows, while *aa* are fixed points and *b* a spring point which comes into action when the car travelling left to right arrives at *X*; a spring point could be provided at *Y* also. A very satisfactory form of turnout is shown in fig. 1046, and is known as the straight

turnout; and by reason of the cars at each end being led straight in, a marked smoothness of working results from this design. For a cross-over road, as in fig. 1047, hand-worked movable points are used: a cross-over road at the

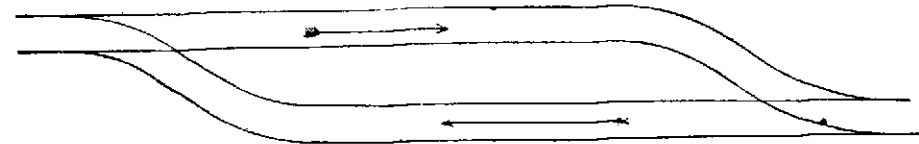


FIG. 1046.

end of a double track in fig. 1048, which, however, would not be suitable to a line where two cars were coupled together, the second being merely a trailing car; and in such cases, in order to get the trailer at the rear end for

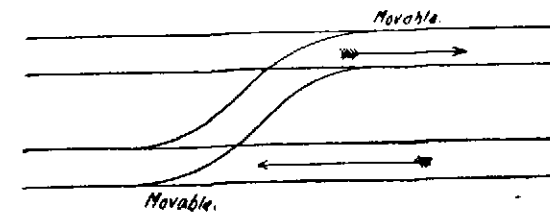


FIG. 1047.

the return journey, a second cross-over road about 100 ft. from the first would have to be provided. Such work is, however, more applicable to light railways than street tramways. Electrically-operated points for junctions are

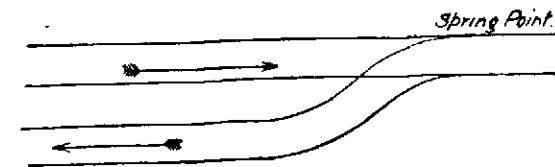


FIG. 1048.

much in use on the Dublin United Tramways, apparently with marked success. The system essentially consists of a tongue, electrically operated, remaining otherwise in one position. Cars which run in the direction that this position

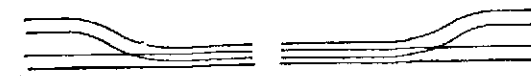


FIG. 1049.

would guide them simply pass on, those which require to go in the other have an arrangement which engages with a lever on the trolley wire. An induced current flows into the point mechanism and shifts it. The tram passes on to its proper track, and another engagement to the trigger puts the

point to normal.* Another system of track laying is by means of inter-lacing lines, as in fig. 1049. Their use is confined to short lengths of narrow streets through which double tracks pass. Their use avoids points, but otherwise renders that piece of track "single." Again, the wear on the lines is only half what it would be if they were single, an important consideration where populous districts and narrow streets are concerned, to avoid having the streets "up."

The actual paving of the tramway tracks deserves attention. The usual system is granite setts, although wood is used in many cases, but is not satisfactory. It is usual for the tramway body to repair the roadway between the lines and 18 in. on either side. The setts next to the rails will be found to wear down very quickly. This is because the ordinary road traffic persists, because of the consequent ease of traction in keeping to the rails. They must be set right up flush against the rail very carefully on a firm foundation, and many engineers prefer to use cast-iron paving blocks next to the rails, which, especially if they are chilled, make a very satisfactory and lasting job. They usually have a checkered surface. Where the road is part setts and part macadam, the macadam will always want attending to if the road is to be maintained in anything like working order. Tramway paving is a frequent source of trouble to municipal engineers. On the one hand, those responsible for tramway administration say that it is unfair that their undertaking should have to bear the first cost and maintain about 16 ft. of paving in the centre of the roadway, which is not worn out by their electric cars, and which is used by general vehicular traffic, and in many instances by competing forms of locomotion. On the other hand, the road authority claims that the presence of tramway lines makes it necessary for paving repairs to be carried out much more frequently, and the tendency is for vehicular traffic, where the tramway service is very heavy, to be concentrated at the side of the road, or in cases where the tramway service is light, for the vehicular traffic to concentrate on the tramway track and cut grooves in the paving outside the 18-in. margins. It is also argued by the road authority that the tramways undertaking is rightly charged with the cost of the paving and its repair in the tramway area as a small payment for the right to use the street, but it will be noticed that no suggestion is made that competing forms of locomotion should be charged for a similar concession.

For instance, take the following facts:—

- (1) The local road authority can stipulate what paving is to be used.
- (2) Competing forms of locomotion do not pay for the cost of installing or maintaining the paving.
- (3) The tramway authority has to pay rates on the earning capacity of its lines, whereas other competing forms of locomotion using the streets are not so rated.

With regard to these points, it seems altogether wrong that where the tramway authority has to bear the first cost and also the maintenance of the paving in the tramways area, the road authority has power to stipulate the class of paving to be used, especially as the softest and the shortest-life paving is generally selected. The fact that wood and soft-granite paving require constant attention and renewal, and consequent interference to all classes of traffic, appears to be quite lost sight of. In half the cases wood paving is laid because it is an expensive luxury, and one which one district thinks it ought to have because another district has it. If it were not for the

* Messrs Tierney and Malone are the patentees.

tramways, the local authority in certain cases could not probably afford to have it.

Where a horse omnibus or motor omnibus service, however, at the present time uses the street, the owners simply have to pay rates on the stables or depots, whereas the tramway undertaking, in addition to bearing the first cost and the cost of upkeep of the paving in the tramway area, has to pay rates based on the earning capacity of the line; and although the omnibuses cut the roads to pieces, the one half of the capital and maintenance charges falls on the ratepayers in general, a few of whom may use the vehicles in question, and the other half falls on the tramway authority, and indirectly on the tramway passengers. It really comes to this, that the ratepayers give a subsidy to omnibus companies so that they may compete with the tramway

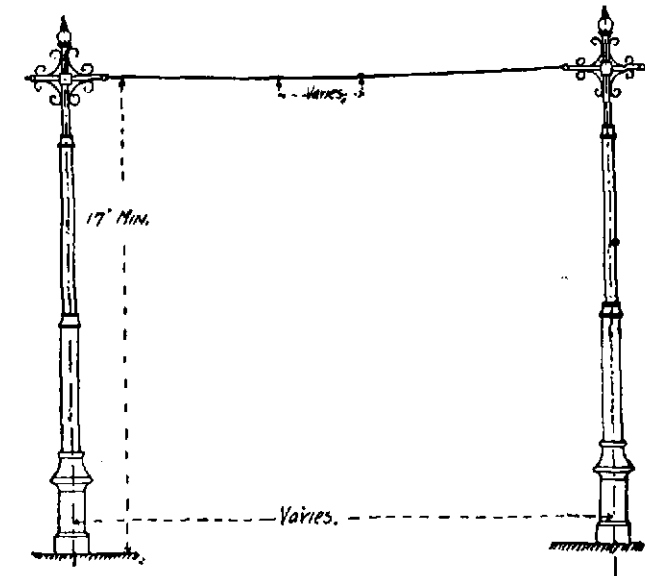


FIG. 1050.

undertaking, which if owned by the municipality would relieve the rates, whereas the omnibus companies do nothing to assist in this direction.

We now come to briefly consider the means whereby the actual distribution of the power is effected.

There are three main divisions:—

1. The overhead conductor.
2. The conduit system.
3. The surface contact system.

By far the most general, and at the same time the cheapest, most economical, and generally satisfactory system, is that of overhead conductors. The actual conductor is popularly known as the "trolley wire," and is insulated from the earth throughout its length, and connected only to the positive terminal of the dynamo or other source of supply. The wire generally used is copper (hard-drawn No. 0), and the P.D. between rails and trolley wire is fixed at 500–550 volts.

The trolley wire is supported in a variety of methods, the two most common being—

1. Span wire, fig. 1050.
2. Bracket arm, fig. 1051.

--while various other expedients are resorted to in particular cases, the most common being attachment to existing buildings,—a system which must only be adopted when due inquiries have been made into the safety of the walls of the building in question to withstand the strain.

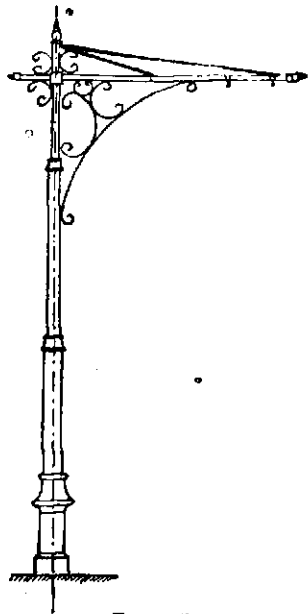


Fig. 1051.

Let us consider what happens in a trolley wire when suspended between its points of support. It sags. This sag takes the form of a catenarian curve correctly; for the purposes of calculation, engineers treat it as parabolic (*vide* Chapter XI.). The usual distance between the supporting pieces, which are specially made insulated fittings, is 120 ft. It may here be remarked that those fittings must be specially well designed, as the maximum allowable current leakage is $\frac{1}{100}$ ampere per mile. Returning to our question of strain, it is usually found in well-fitted lines that the max. sag will be 10 in. in the 120 ft. referred to, under ordinary conditions. However, a high wind will cause considerable strain.

Let L = distance the poles are apart in feet.
 l = true length of trolley wire between the poles in feet at temp. t .
 d = dip of wire in feet.
 w = weight of wire in pounds per foot.
 f_t = tension in the wire at the poles.
 T = maximum temperature of wire.
 K = coefficient of expansion.

Then $d = \frac{L^2 w}{8f_t}$; and $f_t = \frac{L^2 w}{8d}$; (513)

$l = L + \frac{8d^2}{3L}$; and $d = \sqrt{\frac{3Ll - 3L^2}{8}}$ (514)

If the temperature change, then its length and the dip will alter; and if l_1 = altered length, and d_1 = the altered dip, then

$l_1 = l\{1 + (T - t)K\}$; (515)

and $d_1 = \sqrt{\frac{3Ll_1 - L^2}{8}}$ (516)

Some engineers favour the span-wire construction, because the trolley wire can be placed directly over the cars, known as the under-trolley system, while with bracket arms this is not always possible, entailing the use of the side-trolley system. The merits and demerits of each are best left in the hands of electrical engineers to discuss. Then, again, we have the central-post

system, which is generally considered as the most satisfactory of them all.

The posts are usually of lap welded steel in preference to cast iron, because the immense strain set up in these posts would cause cast iron to be a very unreliable material. The general practice is to make them in three sections, fitting telescope fashion into each other. They usually overlap each other about 18 in., and are shrunk together while hot and under pressure, the small cast-iron rings being solely for ornamental purposes. Starting from the bottom we have three sections—

1. 16 ft. long, 7 in. inside diam. 48 in. thick
2. 8 ft. 6 in. " 6 " " " 38 "
3. 7 " 9 " " 5 " " " 30 "

—which applies to medium-sized posts. Some are smaller and some larger than this, according to standard adopted. The arrangement is shown in fig. 1052. In addition we have a C.I. base of design according to architectural requirements, through which the post passes to a heavy concrete base and a light cast-iron cap or finial.

The bracket arms are of hydraulic steel tube. They vary in length, and are not less than $\frac{1}{2}$ in. thick. A tail length of 18 in. is also provided. The two poles are securely fixed to a cast-iron clamp and bolted to the posts, as shown in fig. 1053. The ends terminate in small finials. The usual scroll-work is included according to design, and a wrought-iron stay-rod. They are tested as follows:—

Posts selected at will by the engineer are dropped butt end downwards on to a hard substance 6 ft., and no telescoping, loosening of joints, or other injury should be apparent. Also the post should be supported as a cantilever from the butt end 5 ft. 6 in. At 18 in. from the

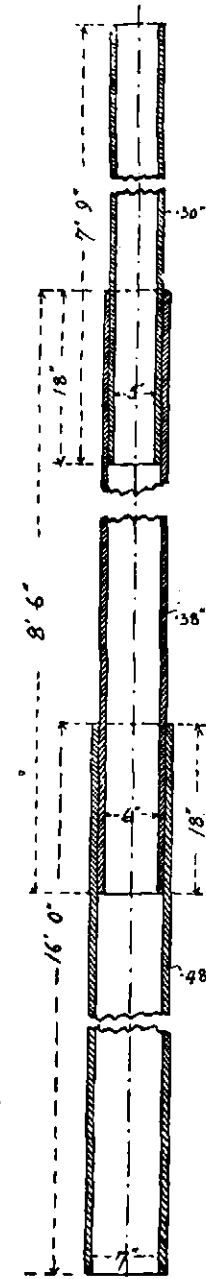


Fig. 1052.

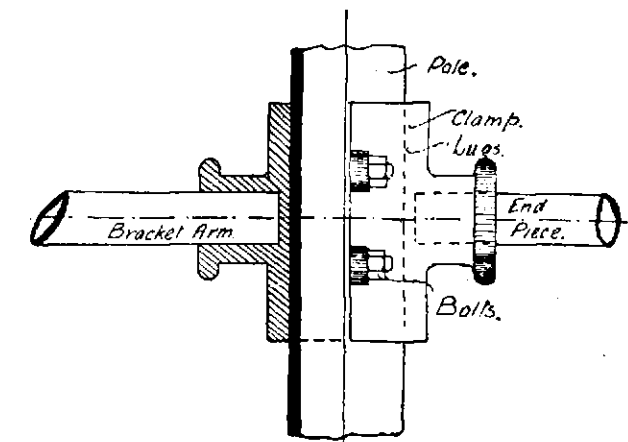


Fig. 1053.

top end a load of 700 lbs. (for a medium-size post, as shown in fig. 1052) should be applied. This arrangement is explained diagrammatically in fig. 1054. The max. deflection must not exceed 6 in., and no permanent deformation should be visible on removal.

The sinking of the posts demands minute attention from the engineer. A hole 5 ft. 6 in. deep (or deeper if required in bad ground) is dug, not larger than absolutely necessary. If the ground is soft, a flagstone is placed for the pole to rest on. The remainder of the hole is filled with 6-1 concrete only, no earth being replaced. It is usual in practice also to set them to a rake outwards from the rails, to allow of their being quite perpendicular when the strain of the wire comes upon them. A deviation of 4 in. at the top of the post has been found ample by the writer, while those posts which terminate a length of line require about 9 in. or 10 in. The fixing of the actual wires is the electrical engineer's work. Round curves they are held in position by steel pull-off wires. The minimum height of the trolley wire from ground-level is 17 ft.; 21 ft. is usually adopted, to allow the use of double-deck cars. In addition to these requirements of distribution, we also have to

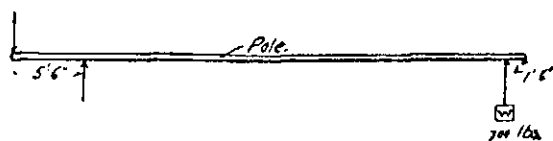


FIG. 1054.

provide for guard wires if there are any telegraph or 'phone wires in the vicinity.

The following is an abstract from the standing orders of the B.O.T.:-

EXPLANATORY MEMORANDUM.

Note.—The expression "telegraph wire" includes all telegraph and telephone wires.

For the purpose of this memorandum, telegraph wires are divided into two classes, namely—

- (a) Wires weighing less than 100 lbs. per mile.
- (b) Wires weighing 100 lbs. or more per mile.

Each guard wire should be well earthed at one point at least, and at intervals of not more than five spans. The resistance to earth should be sufficiently low to ensure that a telegraph or telephone wire falling on and making contact with the guard wire and the trolley wire at any time will cause the circuit breaker protecting that section to open.

The earth connection should be made by connecting the wire through the support to the rails by means of a copper bond. When first erected, the resistance to earth of the guard wires should be tested, and periodical tests should be made to prove that the earth connection is efficient.

Guard wires should be, in general, of galvanised steel, but in manufacturing districts, in which such wires are liable to corrosion, bronze or hard-drawn copper wires should be used.

The gauge of the guard wire should not be less than seven strands of No. 16 or one of No. 8 wire.

The supports for the guard wire should be rigid, and of sufficient strength for their purpose, and at each support each guard wire should be securely bound in or terminated.

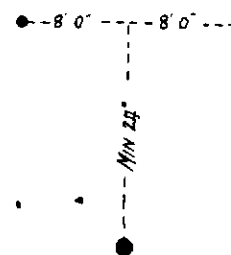


FIG. 1055.

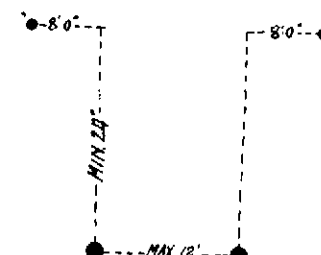


FIG. 1056.

The rise of the trolley boom should be so limited that if the trolley leaves the wire it will not foul the guard wires.

The general arrangements are shown in figs. 1055-1060.

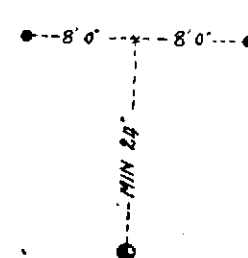


FIG. 1057.

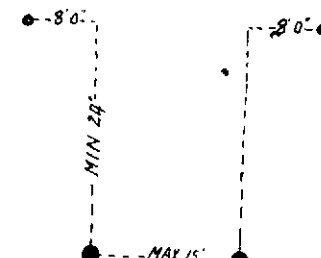


FIG. 1058.

We proceed to review, briefly, underground conduit systems. Though by dint of perseverance, which is so characteristic of the British engineer, the system is now very reliable; yet its expense will, for the present at least, pre-

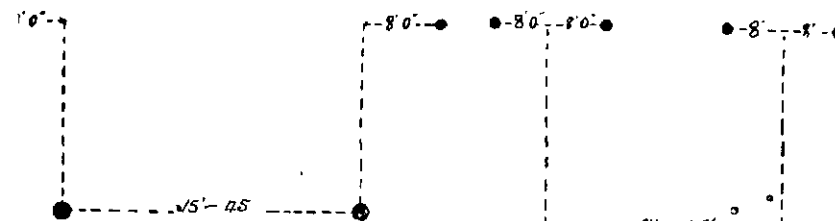


FIG. 1059.

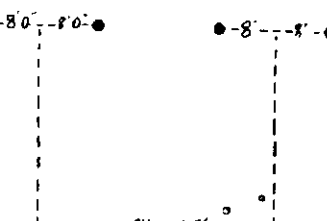


FIG. 1060.

vent its general adoption. The system briefly consists of a tube below the level of the track in which the conductor is laid, and through which access is gained by the collector by a slot $\frac{3}{4}$ in. wide. The size of tube varies, the usual being 16 in. x 24 in.

The principal difficulty which besets the engineer is the system of pipes, etc., very often complicated, which come to view on opening up the ground, which in all cases have to be diverted. The unfortunate part is that, however accurate the thorough surveyors' maps are, the depth of small water and gas mains is rarely obtainable; hence, estimating is very difficult. It is stated on reliable authority that the cost of this system for double track lies between

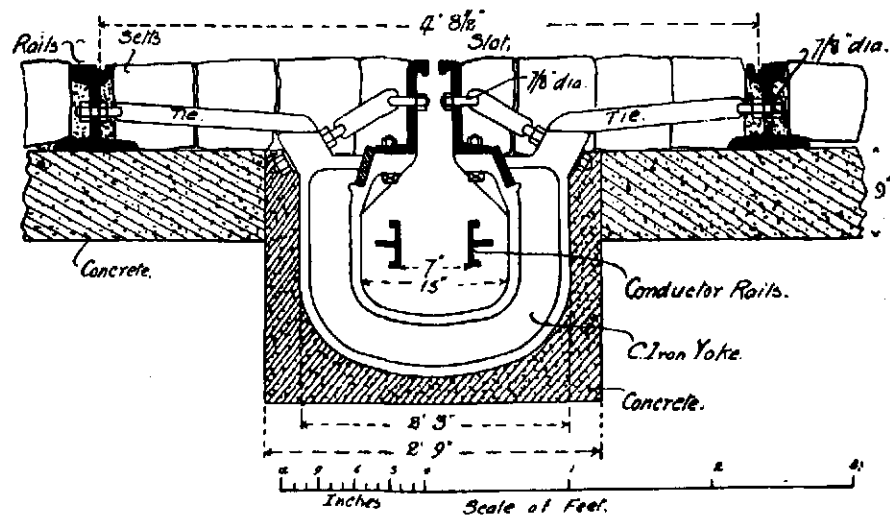


FIG. 1061.

£26,000 and £32,000 per mile. It may be mentioned, also, that side slots are also used, but find disfavour by some, because the slot has to be large enough for the wheel flange to run in, causing a considerable gap on the points, which are very dangerous. This has led to a combined system—side-slot on straight lines and centre-slot for points.

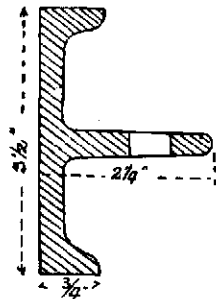


FIG. 1062.

Fig. 1061 shows the London County Council system, which is fairly typical, and the soft-steel conductor rails in fig. 1062. The conduits want very careful draining. On gradients this is obviously easy; on the straight, however, sumps are sunk every 120 ft. or so, the conduits falling both ways.

The surface-contact system is little used in England, at the time of writing Wolverhampton being the only city to adopt it. Briefly, the system consists of studs on the roadway, about 3 in. in diameter, placed at 10–16 ft. centres, and projecting when in action about

half an inch. The distance apart will, of course, be limited by the smallest cars used.

For description, the writer has chosen the Lorraine system, as being of greatest interest to British engineers, being in working at Wolverhampton. The two outer skates are magnetised, while the centre one, made of phosphor bronze, is suspended from a wooden bar by means of a piece of

rubber. It will be seen from fig. 1063 that the cast-iron studs have nickel-steel centres: when the skate passes over the stud, the lower carbon contact

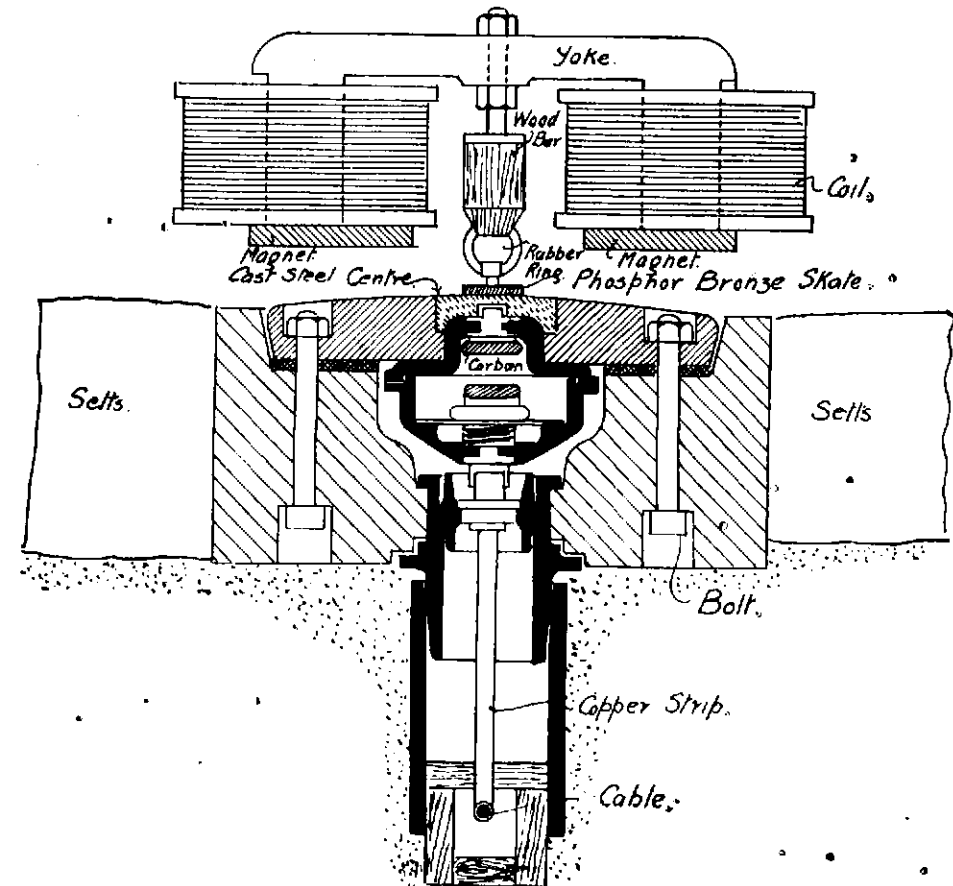


FIG. 1063.

and the armature are raised; thus connection with the upper contact is made.

APPENDICES.

APPENDIX I.

A VERY frequent duty of the civil engineer, especially those who serve public bodies, is the preparation of plans of work to be sanctioned by Parliament or the Local Government Board, the latter principally in the case of loans repayable over a certain number of years. Like similar engineering matters, there is much law to be considered; and although an engineer certainly benefits by reason of having a sound knowledge of legal matters, this being a book on Engineering, we have avoided altogether legal aspects, though there is a great temptation to include them. The following are the general requirements of the Local Government Board.

Elevations, plans, and sections of structural works should be on a scale of not more than 8 ft. to 1 in. The scale of the general plans should be the same as the horizontal scale of the sections, and the vertical scale of sections of sewers should be of not more than 10 ft. to 1 in. Copies of drawings and plans on tracing-linen are preferable to copies on paper. When it is practicable, plans and tracings are much more conveniently transmitted folded than in a roll.

Specifications should contain clauses as to payments and retention money; term of completion of work; penalty for default; extras and alterations (*orders for which should, in all cases, be specified to be made by the Council*); subletting; annulling of contract for insufficient progress; responsibility of contractor for damages by trespass or otherwise; term of maintenance; and settlement of disputes.

In the case of laying pipes or sewers, inspection and testing previous to the work being covered up should be provided for.

In the case of cast-iron pipes under pressure, the specification should require that they should be actually tested. The weight of pipes is best specified by stating the weight of a pipe-length (in the work) for each size.

Cement should be subject to specified tests for strength, fineness, and weight.

The joints of pipe sewers should be made with cement mortar, and the specification should provide for thoroughly clearing the pipes after jointing.

REGULATIONS FOR SEWERAGE WORKS.

I. A map of the district to be sewered, on a scale of not more than 220 ft. to 1 in., showing the dimensions and course of all proposed sewers, positions of man-holes, ventilators, and gully-traps, the inclination of each sewer or pipe sewer from point to point, and the heights at each of such points of the surface of the streets, and of the lowest interior part of the sewer or pipe sewer, both expressed in feet and decimals, over a fixed common reference datum point, referred to Ordnance datum.

II. The course, dimensions, and levels of existing sewers, and information as to how they are to be dealt with.

III. In case of a discharge into tidal waters being proposed, a map of the district adjoining the outfall, corrected to date as to new buildings. The lines and levels of

high and low water, springs and neaps, should be shown, and any information available as to outside soundings and currents should be given.

N.B.—The consent of the Board of Trade must be obtained to the execution of any works below high-water mark.

IV. A statement of the area contributing to each outfall, with its population.

V. Any special duties likely to be thrown on the sewers from factories.

VI. Is there a probability of a neighbouring district seeking the use of the proposed sewers? Full particulars as to area, population, etc., are to be given.

VII. Are any portions of the proposed works to be situate outside the district?

VIII. In what manner and where the sewage is to be disposed of, having regard to the provisions of the 19th section of the P. H. Act 1878, which forbids any authority to make or use any sewer, drain, or outfall for the purpose of conveying sewage or filthy water into any natural stream or watercourse, or into any canal, pond, or lake until such sewage or filthy water is freed from all excrementitious matter, or other foul or noxious matter, such as would affect or deteriorate the purity in such stream or watercourse, or in such canal, pond, or lake, also observing the provisions of the Rivers Pollution Prevention Act, 1876, 39 and 40 Vict. cap. 76.

SPECIAL REGULATIONS FOR WATERWORKS.

I. The number of inhabitants in the district it is proposed to supply with water.

II. The source from which the water is to be taken and the quantity it will supply, with the method by which such quantity has been ascertained.

III. The working head in ft. over the highest point to be supplied in the district.

IV. A copy of the analysis of the water. (An analysis must be obtained in each case.)

V. The quantity of water which it is proposed to supply daily for drinking and domestic purposes.

VI. The quantities to be supplied for other purposes, specifying same.

VII. An Ordnance map, on the scale of 6 in. to 1 mile, showing the source, position of the works, the catchment area, and the course of the mains.

VIII. A longitudinal section along track of supply main, referred to Ordnance datum.

IX. A map of the district to be supplied on a scale of not more than 220 ft. to 1 in., showing the position and diameter of distributing pipes, positions of fountains, and a sufficient number of heights of the surfaces of the streets over Ordnance datum, to enable a judgment to be formed of the efficiency of the scheme. If any existing pipe-supply is in use, the works and pipes thereof should be shown.

X. If the district is supplied by any water company, full particulars should be given of the constitution and rights of the company, and the supply afforded by them.

XI. Any available information as to rainfall or gaugings.

XII. Whether the right to take the water has been secured, both as regards (1) the owners and occupiers of land where the supply is taken off, and (2) the riparian owners or occupiers below that point.

XIII. The storage capacity of any reservoir or service tank.

XIV. Are any portions of the proposed works to be situate outside the district?

XV. Any further particulars bearing on the case.

APPENDIX II.

A VERY important duty of an engineer, after designing his works, getting out the quantities, and finally advertising his tenders, is to see that the conditions under which such contract is to be executed are properly drawn up. It is also equally essential that the proper forms of tender be drawn up; and to aid the proper drawing up of such documents when left to the engineer, the writer has added examples of the conditions and forms referred to which have been in use in his office for many years. They can, of course, be altered, abridged, and amplified to suit special cases.

Two shorter forms of tender for small jobs are also given, A and B.

GENERAL CONDITIONS OF CONTRACT.

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„ Setting out Work, and Responsibility for Errors	4
„ Materials, Property therein when delivered	5
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NOTE.—In the following Conditions and Specification of Works, the term “the Employer” is to be held to designate the one party in this contract, viz. for whom the work is to be executed; and the term “the Contractor” is to be held to designate the other party to the contract, viz. by whom the works are to be executed.

The term “the Engineer” shall, in the case of death or inability from any cause of the Engineer whose name is subscribed to the plans and specification, be held to designate one of the following, viz.

or
whom the Employer shall appoint in such case to carry into effect the provisions of the contract; or in case of the inability, or failure of such Engineer, then such Engineer (being a Member of the Institute of Civil Engineers) as the Employer shall appoint.

Plans, specifications, copies thereof, and their custody.

1. One set of copies of the Contract Drawings and Specification, together with such further drawings as may be from time to time required in the further elucidation thereof, shall be provided by the Engineer to the Contractor for his own use, and the same, or copies thereof, are to be kept on the buildings in charge of a competent foreman, who is to be constantly kept on the ground by the Contractor, and to whom instructions can be given by the Engineer.

The Contractor to make all copies, other than those above described, that he may require for his own use, of either drawings, specification, or details: and all copies must be returned to the Engineer before the final certificate is given, and the Contractor shall not be entitled to require such certificate until all said copies shall have been delivered up, or their loss accounted for to the satisfaction of the Engineer.

The Contract Drawings are to be signed by the Engineer and Contractor, and the whole of the works are to be executed in all respects in conformity therewith, and also with such further drawings and instructions as may be hereafter provided for the elucidation thereof.

Conformity with plans and specification.

2. Any descriptions or particulars written on the drawings are to be equally binding as, if contained in the specification, and everything is to be done that is usual and necessary for the completion of the several works comprehending what may be reasonably implied by or inferred from the drawings or specification, although the same may not be specially mentioned.

Should there appear on the face of the drawings any works not described in the specification, or should there be any works described in the specification and not shown on the drawings, they shall be respectively taken as if described and shown in

both drawings and specification, and executed as part of the work contracted for; and in case of any discrepancy between the drawings and specification, the Architect is to decide which shall be followed.

3. The Contractor is, at his own cost and charge, to provide all and every kind of scaffolding, platforms, ladders, tools, machinery, and everything necessary to complete the several works hereinafter described or shown on drawings, unless special exemption shall hereafter be made. All plant, shoring, timbering, pipes, scaffolding, platforms, and ladders to be of the most substantial description, and to become the property of the Employer as soon as they shall have been delivered on the ground, and to remain his property until the completion of the works, when all unused timber, pumps, scaffolding, pipes, plant, etc., shall revert to and again become the property of the Contractor, but the Employer is not to be accountable therefor, or be responsible for any loss or injury thereto, nor shall the Contractor be entitled to any payment or instalment on account of such plant.

Plant, and employer's temporary property therein.

4. The Contractor is to set out the whole of the works, and during the progress of the works to amend, on the requisition of the Engineer at any time, any errors which may arise therein, and, upon request, is to provide the necessary appliances, or furnish the necessary vouchers to prove that the several materials are such as are described. For setting out the work, figured dimensions, where given on the drawings, are to be taken in preference to measurement by scale.

Setting out work, and responsibility for errors.

5. All work and materials brought and left upon the ground by the Contractor, or by his order, for the purpose of forming part of the works, are considered to be the property of the Employer, and are not to be removed or taken away by the Contractor or any other person without the special licence and consent of the Engineer, but the Employer is not to be in any way answerable for any loss or damage which may happen to or in respect of any such work or materials, either by the same being lost or stolen, or injured by weather, or otherwise. All the materials and workmanship are to be the best of their respective kinds, or such as the Engineer shall approve as suitable for the intended purpose.

Materials, property therein when delivered.

6. The Engineer is to have full power to require the removal from the premises of all materials which, in his opinion, are not in accordance with the specification, and in case of default the Employer is to be at liberty to employ other persons to remove the same, without being answerable or accountable for any loss or damage that may arise or happen to such materials; and the Engineer is also to have full power to require other proper materials to be substituted, and in case of default the Employer may cause the same to be supplied, and all costs which may attend such removal and substitution are to be borne by the Contractor.

Defective materials and workmanship, etc.

Should any of the works be, in the opinion of the Engineer, executed with improper materials or defective workmanship, the Contractor is, when required by the Engineer during the progress of the work, forthwith to re-execute the same, and to substitute proper materials and workmanship; and in case of default of the Contractor in so doing within a reasonable time, the Engineer is to have full power to employ other persons to re-execute the work, and the cost thereof is to be borne by the Contractor.

Any defects, subsidences, shrinkage, and other faults which may appear within months from the completion of the building, and arising out of defective or improper materials or workmanship, are, upon direction of the Engineer, to be amended and made good by the Contractor at his own cost, unless the Engineer shall decide that he ought to be paid for the same; and in case of default, the Employer may recover from the Contractor the cost of making good the works.

Should the Engineer see good and sufficient reason either for the facilitating of the work or for any other cause or circumstance arising in the course of the contract, he shall have power to sanction the substitution of a description of material or workmanship different from or less costly than that specified; or should any material or workmanship be found at any time executed, without the special sanction of the Engineer, of a quality or in a manner inferior to what is specified, to permit such to remain rather than require its removal, and in such case to deduct what he may consider is equivalent to the difference in value between the work as executed and as specified from the amount of any certificate.

Clerk of
Works.

12. The Engineer shall have power, with the consent of the Employer, to appoint a Clerk of Works, in which event he shall be paid by the Employer, and shall, if the Engineer direct, have the custody of the copies of the contract drawings and specification, and of all detail drawings and written instructions issued by the Engineer during the progress of the works, to which he shall give the Contractor and his workmen access at all reasonable times, and all such copies are to be the property of the Engineer and to be returned to him on the completion of the works.

A book shall be kept, if the Engineer so direct, by the Clerk of Works on the premises, in which an entry shall be made by the Contractor, or some person on his behalf, of all extras, additions, omissions, and deviations from the contract duly ordered, as herein set forth, and all measurements for same, if made by the Clerk of Works, shall be conjointly taken by the Contractor or some person on his behalf, and signed by them respectively, and the value thereof is to be ascertained and paid for in the manner hereafter provided for the ascertainment and payment of extra works, and not otherwise.

The appointment of a Clerk of Works shall not remove or in any respect alter or modify the responsibility of the Contractor for errors arising in the work under clause 4.

Engineer's
control and
judgment.

13. The Engineer is to have at all times access to the works, which are to be entirely under his control. He may require the Contractor to dismiss any person in the Contractor's employ upon the works who may be incompetent or misconduct himself, and the Contractor is forthwith to comply with such requirement.

Engineer's
decision.

14. The decision of the Engineer on all questions that may arise relating to the execution of the works, the meaning of the drawings and specification, the manner of the execution of the works, or the quality of materials, the nature, quantity, and quality of the works executed, or to be executed, or what amount was included in the contract, is to be final, conclusive, and binding.

Payments.

15. The Contractor is to be paid as the work proceeds, on the written certificate of the Engineer, in amounts at the discretion of the Engineer, but in no case shall any such amount added to the amounts of former certificates exceed seven-eighths of the value of work done and material delivered and remaining on the ground up to the date of such certificate. The balance, less any deductions made under clause 7, shall be payable, possession of the building having been given up to the Employer, months after the date of the Engineer's final certificate. The Contractor shall not be at liberty to demand any payments from the said Employer, nor shall the said Employer be bound to pay any sum whatever for contract or extra works without production of said certificates, and then only for the amounts therein specified.

Payments on account are not to be considered as admitting that the work done is accepted, and it is therefore to be understood that the Engineer shall have power during the progress of the works, and before the signing of the final certificate, to recall and thereupon to modify as may be necessary or proper his last or any former certificates.

Responsi-
bility for
damages.

16. The building, from the commencement of the works to the completion of the same, is to be under the Contractor's charge, who is to be held responsible for, and is to make good, all injuries, damages, and repairs occasioned or rendered necessary to the same by collapse or by causes over which the Contractor shall have control, and he is to hold the Employer harmless from any claims for injuries to persons or for structural damage to property happening from any neglect, default, want of proper care, or misconduct on the part of the Contractor or anyone in his employ during the execution of the works. The Contractor shall also pay and make good to the Employer all sums which the Employer shall become liable to pay under and by virtue of the Workmen's Compensation Act 1897, the Employers' Liability Act 1880, or any Act amending the above, or by the Common Law, as and for compensation for personal injury caused by accident during the continuance of this contract to any workmen while engaged in the works the subject-matter of this contract, and whether in the immediate service of the Contractor or in that of a Sub-Contractor, or person executing work for the Contractor under this Contract.

Employer's
right of
access.

17. The Employer is at all times to have free access to the works, and is to have full power to send workmen upon the premises to execute fittings and other works

not included in the contract, for whose operations the Contractor is to afford every reasonable facility during ordinary working hours, provided that such operations shall be carried on in such a manner as not to impede the progress of the works included in the contract, but the Contractor is not to be responsible for any damage which may happen to or be occasioned by any such fittings or other works.

18. If the Contractor shall become bankrupt, or compound with or make any Bankruptcy assignment for the benefit of his creditors, or shall suspend or delay the performance of his part of the contract (except on account of causes mentioned in clause 7, or on account of being restrained or hindered under any proceedings taken by parties interested in any neighbouring property, or in consequence of not having proper instructions, for which the Contractor shall have duly applied), the Employer may give to the Contractor, or his assignee or trustee, as the case may be, notice requiring the works to be proceeded with, and in case of default on the part of the Contractor of his assignee or trustee for a period of 21 days, it shall be lawful for the Employer to enter upon and take possession of the works, and to employ any other person or persons to carry on and complete the same, and to authorise him or them to use the plant and materials upon the works, and the cost and charges incurred in any way in carrying on and completing the said works are to be paid to the Employer by the Contractor, or may be set off by the Employer against any money due or to become due to the Contractor.

19. No portion of the works shall be sublet without the consent of the Engineer, subletting given in writing, and the party or parties appointed to do same are to be subject to his approval; and the Contractor is to be responsible for his or their proper execution of the works so sublet in strict conformity with the provisions of this contract.

20. In the case of any sum or sums of money being provided by the specification for any work or materials, the Engineer shall be at liberty to deduct from the amount of the contract, or apply such sum or sums as he shall think fit without any deduction whatsoever, and where articles are specified to be of a particular cost, P.C., the Engineer shall be at liberty to call upon the Contractor to show to his satisfaction that such sum has actually been expended on the prime cost of such article by the Contractor; and in case of his failure to do so, the Engineer shall allow the Contractor only such sum as he shall think fit for such work, and his decision thereon shall be final and conclusive.

Should the amount of the deductions (save and except for money provisions) from the works contracted for exceed the amount of the extras, the Contractor shall be entitled to be paid 15 per cent. on the excess of said deductions.

21. When the works are to be executed in a city or town, or in such a position that any adjoining or abutting buildings are liable to injury or trespass by the carrying out of the same, the Contractor must be responsible for all such injuries or trespass to persons or property so situated, and must make good same at his own cost and charge. He must also at his own cost serve such notices on the owners and occupiers of such property as the law in such case directs, and must take the liability for any neglect on his part to serve the aforesaid notices. He must also take out and uphold all requisite licences, and pay for same, and must be responsible to the public authorities for obstructions to the public thoroughfare caused by carrying out the works.

FORM OF TENDER.

_____ of _____ in the
County of _____ hereinafter called
"The Contractor," proposes to and hereby agrees with the _____
Council, hereinafter called "The Council," to _____
according to the Specification hereunto annexed, for the sum of _____

Contractor hereby agrees with, and binds himself, his executors, assigns, and administrators, to the said Council to complete and finish said works on or before the _____

Protection
of works
from
weather—
suspension
of same.

7. The works to be protected during frost, snow, or heavy rains as may be directed; and if the Contractor neglects to protect them as directed, the Engineer may do so at the Contractor's expense, and deduct the cost from his next certificate.

Should it, in the opinion of the Engineer, appear necessary at any time to suspend the works or any part of them on account of weather or other circumstances, the Architect shall have power to do so, and to award such extension of time for the completion of the works as he may think reasonable for such delay.

Completion
of works.

8. The Contractor is to commence the works within seven days after the signing of the contract, and to complete the whole (except such other works as the Engineer may desire to delay) within calendar months after the commencement of the same, unless the works be delayed by reason of extra works required, in the opinion of the Engineer, extension of time, any inclement weather or causes not under the Contractor's control, or in case of a combination of workmen or strikes, or lock-out affecting any of the building trades, for which due allowance shall be made by the Engineer, and then the Contractor is to complete the works within such time as the Engineer shall consider to be reasonable, and shall from time to time in writing appoint; and in case of default, the Contractor is to pay or allow to the Employer, as and by way of liquidated and agreed damages, the sum of £ : : per week for every week during which they shall be so in default, until the whole of the works (except as aforesaid) shall be so completed, provided the Engineer shall in writing certify that the works could have been reasonably completed within the time appointed.

The works, on their completion, are to be left in a clean, complete, and perfect state, and all rubbish which may be accumulated on the site during their progress is to be removed by the Contractor, unless otherwise directed by the Engineer; and the Engineer is at liberty to refuse his final certificate until these requirements are complied with.

A certificate of the Engineer, showing the final balance due or payable to the Contractor, shall be conclusive evidence of the works having been duly completed, but without prejudice to the liberty of the Contractor under the provisions of clause 11.

In case the works shall not be carried on with all proper despatch, so as to afford, in the opinion of the Engineer, a reasonable probability of their being completed within the period stipulated in the contract, and which he shall certify in writing, the Employer shall be at liberty, on giving ten days' notice in writing, signed by him and left in the Contractor's usual place of business, to enter upon the works, take possession of same, and employ such other persons, either by contract or otherwise, as he shall deem proper, and shall be at liberty to use all scaffolding, plant, and materials that may be upon the ground to complete the buildings, and to provide all necessary materials and workmanship for same, the cost of which and of any deficiencies shall be ascertained by the Engineer and deducted by him as liquidated damages from any moneys, including the drawback due to the Contractor on foot of Contract or extra works; and should there be no balance due on either contract or extra work, the Contractor shall be liable for same.

Deviations,
omissions,
variations.

9. Deviations (by omission, addition, or variation) from the work as set forth by the contract plans and specification, duly sanctioned by the Engineer, shall not vitiate or annul the contract.

The Engineer shall have full power during the progress of the works to make alterations in the works as set forth in the plans and specification which do not involve additional cost. Any alteration which involves additional cost is to be borne by the Contractor unless he shall have, previously to its execution, obtained the written order of the Engineer for its execution.

The Contractor is not to vary or deviate from the drawings or specifications, or execute any extra work of any kind whatsoever, unless upon the authority of the Engineer, to be sufficiently shown by an order in writing, or by any plan or drawing expressly given and signed or initialed by him as an extra or variation, or by any subsequent written approval signed or initialed by him. In case of day work, all vouchers for the same are to be delivered to the Engineer (or clerk of works) at latest during the week following that in which the work may have been done, and

only such day work is to be allowed for, as such, as may have been authorised by the Engineer to be done, unless the work cannot from its character be properly measured and valued.

No allowance shall be made for any work extra to the contract unless same shall have been executed by the authority of the Engineer upon an order, as hereinbefore mentioned; such orders specifying that such work is to be considered as an extra. A statement in writing of all work which the Contractor may consider extra to the contract, which may be done or be in progress at the date at which application is made for a certificate, shall be handed by the Contractor to the Engineer at the time of making such application. Any such work which may be executed before the granting of the first certificate, or between the dates of any two consecutive certificates, but which shall not be so returned when applying for the certificate next following, shall not be considered as work extra to the contract.

10. The prices in the detailed estimate furnished by the Contractor to the Engineer marked shall be the values of all additions, omissions, or variations, unless at any time it should appear that such prices have not been fairly made up, and, as a schedule of prices, shall stand as a part of this contract; but the Employer shall not be responsible for omissions of items or accuracy of the quantities.

Should it appear on the face of such detailed estimate that any deduction has been made from the gross total of the estimate or any section thereof, by way of percentage, so as to reduce the net sum or sums to the contract amount, each rate in detail in being applied for calculating the value of additions, variations, and omissions, shall be subject to reduction by a corresponding percentage.

Where the schedule of prices does not apply, the Engineer is to fix such rates as he shall consider proper, basing them upon the schedule prices; and should a schedule of prices not be made a part of the contract, such prices as he shall consider fair.

The Engineer shall have power, at any time, to appoint a Building Surveyor to check or measure-up works, a statement of which may have been returned by the Contractor from time to time, as before provided, and which shall be admitted to be works extra to the contract, or to measure-up works which may be omitted or altered from time to time; and at the completion of the work the Engineer shall have power to appoint the same or another Surveyor to prepare a bill of omissions, variations, or deductions, and price the same, as hereinbefore provided.

The fees of the Surveyor are to be borne half by the Employer and half by the Contractor, and the Engineer shall have power to deduct the Contractor's moiety from any moneys due to him on foot of the contract.

11. Should any misunderstanding arise between the Engineer and the Contractor in reference to the quantities or prices of additions, variations, or omissions, duly ordered as herein set forth, the Engineer shall have power to name a Building Surveyor to take out or measure said quantities and price same as hereinbefore provided; and when the Surveyor shall have made up his said account, he shall furnish a copy thereof to the Engineer and Contractor, and if either party shall object thereto, he shall be at liberty to do so within 31 days after the same shall have been furnished, specifying such objections in writing, detailing the items; in which case, unless the said Engineer or Contractor agree thereon, the said items shall be referred to the arbitration of two Engineers—one to be named by the Engineer and one by the Contractor—with power to them to appoint another Engineer as umpire, and the decision of said arbitrators and umpire as to said items is to be conclusive and binding on the said Contractor, the said Employer, and the Engineer; and the account is in like manner to be final and conclusive on the same parties as to all items not the subject of objection as aforesaid, and the Engineer shall give the Contractor a certificate therefor, or, in case of objections, for the amount to be ascertained as aforesaid, and the said Employer shall be bound to pay the Contractor for such extras the sum specified in such certificate and no more, within one week after the production to him by the Contractor of such certificate.

The fees of the Surveyor are to be paid in like manner as before provided, and the costs of arbitration are to be in the discretion of the arbiters and umpire, and to be paid as they or he may direct.

Arbitrating
on valua-
tions.

And should the entire of the said works not be completed within the aforesaid period, the Council shall be at liberty, and the Contractor hereby authorises them, to charge his executors, administrators, and assigns the sum of _____ pounds sterling for every week the said works shall be delayed beyond the time hereby limited, and to deduct the same out of any moneys payable in respect of said works.

And Contractor further agrees and undertakes that his executors, administrators, or assigns will not at any time alienate or sublet the works hereby contracted for, or any part thereof, to any person or persons whatsoever, or substitute or depute any other person to execute the same or any part thereof; but that the works to be executed shall be exclusively and in all respects executed by said Contractor, and under his own direction and supervision.

And on breach of this agreement or any part thereof, Contractor undertakes and agrees, for his heirs, executors, administrators, or assigns, that the said Council shall be at liberty to rescind this agreement altogether, and withhold payment of all sum and sums of money which could or might otherwise have been claimed on account of the works theretofore executed under this contract and agreement; and further, that all right to the said sum and sums of money shall be forfeited by the Contractor, his executors, administrators, or assigns, anything herein contained to the contrary notwithstanding.

Should the rate of progress of, or the mode of proceeding with, or the maintenance of the works be considered, by a duly authorised officer of the Council, at any time unsatisfactory, they shall have power to procure and make use of, for the purpose of the works, all labour, plant, and materials they may think necessary, and the cost of all such plant, labour, and materials shall be deducted from any money that may be or thereafter become due to the Contractor. The mode of proceeding with the works shall at all times and in all respects be subject to the approval of the Engineer or other authorised officer of the Council, but such condition shall in no case be taken as relieving the Contractor from entire responsibility for the safe conducting of the work. The Contractor shall be liable for the works during progress and to the end of period of upholding, and shall further be liable for all contingencies, failures, accidents, or injuries to persons or properties, and shall take all needful precautions, as, in case of all damages, the Council shall be at liberty to make fair compensation for Contractor's default, and deduct the amount thereof from any sum due to the Contractor.

The whole of the work under the Contract is to be of the soundest description. In case the Engineer shall be of opinion that any part of the work is being executed unsoundly, or not with proper materials, or in any respects not in accordance with the plans, specification, or other of his instructions, or to have become unsound or imperfect, or in case the Engineer shall give the Contractor a notice in writing requiring any part of the works to be taken down and removed and to be re-executed properly, the said Contractor shall forthwith comply with such notice, and do the several matters required without extra charge; and in case the said Contractor shall not, within two days after receipt of such notice, have duly complied therewith, the Council shall be entitled to cause the works to be taken down and re-executed, and for that purpose expend such sum as their Engineer may consider proper, and to deduct the amount from the contract sum. Any deductions or additions made by order of the Engineer shall not vitiate the contract.

Should the said Contractor appear to the Engineer to be carrying on the work so slowly and carelessly as to render it impossible to complete the works reasonably near the specified time, the Council may annul the contract and complete the work, using for the purpose any plant and materials they may find on the works, and recouping themselves for any loss from money due on foot of contract or by sale of plant; and if these be insufficient, by recovering the balance from the Contractor and his sureties.

In consideration of the execution and upholding of the works of the contract to his entire satisfaction, the Council shall, on the certificate of their Engineer, pay the Contractor the amount of his contract in such instalments as may be so certified, not exceeding in all, on the completion of the works, 80 per cent. of the contract sum,

less by any penalties and deductions as before provided for, the remaining 20 per cent. being retained as part security for the full performance of the contract, the final instalment being the balance due to the Contractor at the termination of the period of maintenance.

In witness whereof _____ have hereunto set _____ hand this
day of _____ in the year of our Lord One Thousand Nine Hundred and _____
In presence of _____
Accepted on the part, and by order of, the _____ Council this
day of _____ in the year of our Lord One Thousand Nine Hundred and _____

Clerk to the Council.

We _____ of _____
and _____ of _____
in the County of _____ hereby do and each of us doth become Sureties to the
Council for the above-named _____
in the sum of _____

for the due performance of the foregoing Contract.

Witness our Hands the day and year lastly in the foregoing Tender written.

Present:

FIRST SURETY.

NAME _____

RESIDENCE _____

POST TOWN _____

SECOND SURETY.

NAME _____

RESIDENCE _____

POST TOWN _____

A

TENDER FOR

To _____

OF _____

DEAR SIR OR GENTLEMEN,—

I, the undersigned, hereby tender, and should the tender be accepted, agree to enter into a Contract to do and execute to the entire satisfaction of your Architect and Surveyor, or any person appointed or duly authorised by you to superintend and

approve of the work, and in strict accordance with the Specification and Conditions prepared by _____ for the sum of _____ (insert the sum in writing),

As witness _____ hand this _____ day of _____ 19 _____

Signature of Person tendering _____

Business Address _____

Amount of Tender £ : :

B

TENDER for Works to be performed in _____

To _____
OF _____

DEAR SIR OR GENTLEMEN,—

Having examined the Specification and Drawings therein referred to and prepared by your Architect, _____, the undersigned, offer to find and provide all materials and plant, permanent and temporary, and do all work and labour, and pay all fees, cost, and expenses incidental to and necessary for the completion of and maintenance of the works shown on the drawings and described in the Specification and General Conditions of Contract, in the manner, by the time, and subject and in accordance with the several clauses, restrictions, provisions, and stipulations in the said Specification and Conditions of Contract contained, to the entire satisfaction of _____ for the sum of _____ (insert the sum in writing),

In case this Tender shall be accepted _____ hereby offer to undertake to execute an agreement for the due performance and fulfilment of the said Specification, and Conditions set forth in the Conditions of Contract, and find two responsible sureties for the due and satisfactory performance of the whole of the works comprised in the Contract.

As witness _____ hand this _____ day of _____ 19 _____

Signature _____

Business Address _____

Amount of Tender £ : :

APPENDIX III.

URBAN DISTRICT OF _____

CONTRACT.

Conditions referring to Appointment of Clerk of Works.

In these conditions the expression "Architect and Surveyor" or "Architect" shall refer to Mr _____ or the Surveyor and Engineer of the Council for the time being. The person appointed as Clerk of Works will not be permitted to undertake any other employment whatsoever whilst in the service of the Urban District Council and he shall be under the direct instructions of the Engineer and Surveyor to the Council.

He will be required to be on the works during all hours when men at work from morning start to knock off in evening, including overtime if worked, and if at any time he fails to be on the works during above hours he will be liable to instant dismissal.

His duties include the following:—

To see that the works are carried through in accordance with the plans and specification, and that the engineer's directions are strictly complied with. To report at once to the architect any attempt on the part of the contractor or his employees to substitute inferior workmanship or materials to those specified, or to evade in any way the full amount of work shown by the drawings and specification, and he must keep a constant watch on all operations in order to detect any such omissions. Special care must be observed by him with regard to all foundations, proportions and mixing of concrete, freedom of sand from loam, condition of timber used in carpentry and joiner work. Soundness of all made ground over which concrete may require to be laid. To see that all concrete of walls, etc., is thoroughly well rammed in casings, and wetted at each lift. Special care must also be observed in testing all cement that is to be used on the works, that it is in proper condition and up to the specification standard. And finally, to see that the best value is obtained for the outlay, and that thoroughly good work is turned out. He must keep an accurate account of the amount of work completed and report to engineer daily, clearly defining each set of measurements so that nothing may be remeasured or omitted, and to keep a careful account of any omissions of works or extras to contract.

All these measurements and entries to be made in ink in books provided for the purpose, and signed weekly by the Surveyor, Clerk of Works, and Contractor.

APPENDIX IV.*

WHAT CONSTITUTES A "STREET"?

THE general notion of a "street" is that it consists of a roadway with buildings on each side, as distinguished from a "road," which may have buildings on one side only or no buildings. It is clear, however, that legally the term "street" may have a much more extended meaning than this. It is frequently necessary to consider the term in connection with the requirements of the Public Health Acts, which regulate the powers of local authorities as to laying out new streets. On turning to the Public Health Act, 1875, we find the following definition of "street" in section 4 as "including any highway (not being a turnpike road) . . . and any road, lane, footway, or passage, whether a thoroughfare or not." The application of this section was discussed by the High Court recently in a case in which the corporation of a provincial town took proceedings against the owner of property and a builder for contravening one of the municipal by-laws made under s. 157 of the Public Health Act, 1875, relating to the laying out of new streets. By the by-law in question it was provided that where a new street exceeded 100 ft. in length and was the principal means of access to any

* From "Legal Notes" in the *Illustrated Carpenter and Builder*.

building, it should have a width of 36 ft. and a carriage-way 24 ft. wide. Now in the case in question the owner had built some cottages 200 ft. back from a roadway, the only access to them being by a footpath 5 ft. or 6 ft. wide. On the plans being submitted to the borough surveyor for approval he raised the objection that the suggested mode of access would be a breach of the by-law, but the builder, on his own authority only and without reference to the owner, promised that a roadway of the necessary width should be made, and submitted an amended plan. Eventually the plans were approved by the local authority, subject to a condition that the road should be made as required by the by-laws, the builder's undertaking being accepted as to this. The undertaking was, however, not carried out, and an action was accordingly brought by the Attorney-General and the corporation against the owner and builder for a declaration that the by-law had been contravened, and for an injunction calling on them to lay out the street as required by the by-law. The question then to be decided was whether this path was a "street" within the meaning of s. 4 of the Public Health Act, 1875. The defendants argued that "new streets" in s. 157 of the Act, under which the by-law was made, meant street in the common acceptation of the term, viz. with houses on each side, and that it was unreasonable to require a street of the width prescribed by the by-law, which only led up to these cottages. The Court, however, decided that there was no limitation to the expression "new streets," but that it applied to everything that was a street within the meaning of section 4, and that the by-laws were not unreasonable, and had been broken by the defendants. An injunction was granted, but against the owner alone, on the ground that, notwithstanding what the builder had done in acting without the owner's knowledge and authority, yet the builder had not control of the land. The decision is important and should be noted.

APPENDIX V.*

ANCIENT LIGHTS.

PRIOR to the decision in the well-known Colls' case, the law was regarded as settled that the owner of ancient lights was entitled not only to sufficient light for the purpose of his then business, but to all the light which he had enjoyed previous to the interruption sought to be restrained. Lord Chancellor Cranworth said that the right as to ancient lights conferred or recognised by the pre-emption Act—viz. an uninterrupted user for twenty years—was an absolute indefeasible right to the enjoyment of the light without reference to the purpose for which it has been used. Colls' case, however, led to a different view being entertained of the meaning of the right attaching to "ancient lights"; and as the principles laid down in that case are of very great importance, we propose to refer briefly to the judgment of the House of Lords in that now leading case. The action was brought by the Home and Colonial Stores against Mr Colls, a builder, to restrain him from erecting a building in Worship Street, Finsbury, in such a way as to obstruct their ancient lights. The building objected to was erected on the site of old buildings which had been pulled down. It was proposed to erect it to the height of 42 ft., the old buildings it replaced being 19 ft. 6 in. in height. The company complained that the erection would seriously darken and obstruct their ancient lights. To that Mr Colls replied that the amount of light which would be left would be sufficient for the use of the company's premises for ordinary trade. Mr Justice Joyce held that, although the new building would deprive the company of a substantial amount of light, causing them real damage, yet that enough light would be left for the ordinary purposes of the occupancy of the premises as a place of business; and as its selling and letting value would not be affected, the company were not entitled to an injunction. This decision was eventually upheld by the House of Lords. The Lord Chancellor, in the course of his judgment, said:—"The test of the right is, I think, whether the obstruction complained of is a nuisance, and, as it appears to me, the value of the test makes the amount of right

* From "Legal Notes" in the *Illustrated Carpenter and Builder*.

acquired depend upon the surroundings and circumstances of light coming from other sources, as well as the question of the proximity of the premises complained of. What may be called the uncertainty of the test may also be described as its elasticity. A dweller in towns cannot expect to have as pure air, as free from smoke, smell, and noise, as if he lived in the country and distant from other buildings, and yet an excess of smoke, smell, and noise may give a cause for action; but in each of such cases it becomes a question of degree, and whether, in each case, it amounts to a nuisance which will give a right of action." Of course it must be taken that the foundation of this judgment rests upon the finding of fact by Mr Justice Joyce that the buildings of the defendant had not so materially interfered with the light previously enjoyed by the plaintiffs as to amount to a nuisance. The test, then, according to this decision, is whether the obstruction amounts to nuisance which causes a right of action, and not how much light is left, but whether the plaintiff has been deprived of such light as to constitute a current nuisance. The loss must be substantial according to the character for which the premises are used, and unless it amounts to that, there is no cause of action. A given percentage of loss of light is not sufficient, but it must be an interference with the comfortable and convenient occupation of the premises within the notions of citizens in this country. The rule thus laid down has been constantly acted upon since Colls' case was decided, and must be taken as the guiding principle now in actions for alleged obstruction of ancient lights.

APPENDIX VI.*

DRAINS AND SEWERS.

WHAT actually constitutes a drain or sewer is one of the most debatable questions of modern legislature. In fact, at present (1909) it is almost impossible to draw a line, so diverse are the opinions of legal experts. We leave the reader to study the question elsewhere. The following is, however, one of the latest cases, and may be of interest. At the time of writing, however, improvements in the law relating to this question are on foot in Parliament.

BUILDING ESTATE—"DRAIN OR SEWER"—NO USE OF PIPES—WHETHER USER NECESSARY TO CONSTITUTE A SEWER.

This was a motion for a declaration that a line of pipes laid by the plaintiff in a portion of his land, which he had laid out as a road, was not a sewer vested in the defendant Council under the Public Health Act, 1875, or otherwise; and that the defendant Council was not entitled to connect any sewer with such line of pipes; and for an injunction restraining the defendants from trespassing and making such connection. The sewer or drain had been constructed by the plaintiff in accordance with plans and specifications deposited with and approved by the surveyor to the Council, and to the satisfaction of the Council, who admitted they were liable for its repair. The plaintiff was developing a building estate, but at present there were no houses on this particular road, and no drainage had hitherto passed down the pipes in question, which were connected at the east end with a sewer constructed by the plaintiff, and now vested in the defendants. The defendants had served a notice on the plaintiff expressing their intention of connecting a sewer which was vested in themselves, and which ran from the west end of the road, with the pipes laid down by the defendant, at the point where their sewer reached the plaintiff's pipes at his boundary in the road; and they alleged that the pipes in question were a sewer vested in themselves. The plaintiff was apprehensive of damage being done to his land by the exercise of the alleged rights of the defendants over the sewer or drain. The definition of a "drain" by section 4 of the Public Health Act, 1875, is "any drain of and used for the drainage of one building only, or premises within the same curtilage, and made merely for the purpose of communicating therefrom with a cess-

* From "Legal Notes" in the *Illustrated Carpenter and Builder*.

Therefore total stress due to bending and direct compression on the fibre of the bent column on the inner side of curve

$$\begin{aligned} &= p_2 + \frac{p_2 l^2}{K^2} \\ &= p_2 \left(1 + \frac{l^2}{K^2} \right) \\ &= \frac{W}{A} \left(1 + \frac{l^2}{K^2} \right) \\ &\therefore p, \text{ say.} \\ \therefore W &= \frac{pA}{1 + \frac{l^2}{K^2}} \end{aligned}$$

= breaking load if p is breaking stress.

The Rankine-Gordon formula is a modification of the Gordon formula for columns and stanchions, taking in the radius of gyration instead of the least width. The results of numerous experiments showed that the strength was not so well indicated by the ratio of length to least width as by the ratio of length to radius of gyration. The Rankine-Gordon formula is

$$P = \frac{fA}{1 + \frac{l^2}{acr^2}}$$

where P = ultimate load in lbs., f = about two-thirds of compressive strength of the material in lbs. for mild steel, say 42,000. A = area of cross-section in sq. in. l = length of column in inches, a = constant, according to mode of fixing, say 4 for both ends fixed, c = constant, varying with the material, say 9000 for mild steel, r^2 = least radius of gyration squared = $\frac{I}{A}$. For example, take a 14-in. by 6-in. by 57 lb. rolled-steel joist used as a stanchion, 20 ft. long, with fixed ends, the ultimate or breaking load will be

$$P = \frac{fA}{1 + \frac{l^2}{acr^2}} = \frac{42000 \times 16.76}{1 + \frac{(20 \times 12)^2}{4 \times 9000 \times 1.67}} = \frac{703920}{1 + 958} = 359510 \text{ lbs., and } \frac{359510}{2240} = 160 \text{ tons.}$$

The maximum safe dead load $\frac{160}{4} = 40$ tons.

Likewise we proceed to investigate in fuller detail the equation

$$\frac{W}{A} \pm \frac{M}{Z}$$

Let a member of a structure (say a steel joist) be subjected to a longitudinal thrust = W , and be also supporting a load which produces a bending moment M at any chosen section.

Let I = moment of inertia of the section of joist,

y = distance of either extreme fibre of cross section from the neutral axis (which passes through the centre of gravity of the section);

p = unit stress at the extreme fibre produced by the bending moment;

then it is proved in any book on structural mechanics that

$$p = \frac{M}{I} y.$$

If

p_1 = unit stress produced by W ,

$$p_1 = \frac{W}{A}$$

where

A = cross-section area.

Therefore total stress at the given point in section

$$= \frac{W}{A} \pm \frac{M}{I} y.$$

The + sign applies when M produces compression at the given point, the minus sign when tension. Many text-books denote $\frac{I}{y}$ by the symbol Z , and term Z "the section modulus," since for any chosen section it is a constant.

It must be remembered that there will be two values for Z , say Z_c and Z_t , the first for the side of the neutral axis in which compression is produced by bending, and the other for the tension side, and so

$$Z_c = \frac{I}{y_c} \text{ and } Z_t = \frac{I}{y_t},$$

where y_c and y_t are the distances of the extreme fibres on the compression and tensile sides of the section. In symmetrical figures, like rolled joists and foundations, this correction will, of course, not be necessary.

APPENDIX VIII.

CONCRETE BLOCKS

MATERIALS.

THE materials most commonly used in concrete block manufacture are sand, gravel, crushed stone, and Portland cement. Much could be written on sands alone, so we will touch upon this subject very briefly. The term is confused by a great majority of concrete-block manufacturers. In concrete work, sand is that material which will pass a $\frac{1}{4}$ -in. screen, and a good sand should be graded from coarse to fine, with the coarser grains predominating. A sand which is well-graded from coarse to fine will make strong concrete. It will also make dense concrete. The strength of concrete varies directly with its density. Let us lay down one fact, however, and call particular attention to it—viz. that no concrete can be stronger than the materials from which it is made.

Many block manufacturers insist on using for facing a very fine sand, mixed in the proportions of one part of cement to two parts of sand, not realising that a coarse, well-graded sand will give as strong a facing with a one to three mixture, and a block of as good appearance as well. They seem to think they must use a fine sand in order to get a smooth block.

When the tamping is directly against the face, as in most machines, the finer particles work themselves through the coarser ones into the face, and thus the surface is just as smooth as though a fine sand were used.

As good a surface can be obtained by using coarse, well-graded sand, and it will be far stronger and more waterproof. The reason for this is, that in a fine sand the grains are nearly all uniform in size, and the percentage of voids will be between forty and fifty per cent., whereas in a well-graded sand the voids may be as low as twenty-five per cent. Consequently, when using the same amount of cement with both sands, the coarser sand will make a coarser, denser, more waterproof concrete.

The fine sand also has a greater number of particles, and, consequently, there is more surface to be coated with the cement mortar. This means that more cement is required.

FACING OF CONCRETE BLOCKS.

The subject of facing concrete blocks is one that well deserves attention. Blocks are usually sold because of their good appearance and satisfactory qualities. The

pool, or . . . with a sewer into which the drainage of two or more buildings or premises occupied by different persons is conveyed. 'Sewer' includes sewers and drains of every description, except drains to which the word 'drain' interpreted as aforesaid applies." By section 13: "All existing and future sewers within the district of a local authority, together with all buildings, works, materials, and things belonging thereto, shall vest in and be under the control of such local authority." It was contended for the plaintiff that a pipe which did not drain anything was not a sewer, and that there must have been user; that to hold that the pipes were a sewer would impose a liability under section 15 on the local authority to repair any pipe laid down in a building estate; and that the case turned on the question whether the local authority were satisfied with the street as sewered. It was contended, *contra*, that it was impossible to say the street had not been sewered, having been laid out under the Council's supervision; that there had been no obligation on the plaintiff to do the work, but that the work, if done, had to be in accordance with the Council's plans; that directly a sewer is made it vests in the local authority; that in *Hornsey Local Board v. Davis* it was held that, although a sewer had never been used, the road was sewered to the satisfaction of the local authority; and that here the Council were satisfied at any rate with the sewer, if not with the street.

The Court decided that it could not grant the application, but that the matter in issue would have to be decided at the time of the action, the costs of the application to be treated as part of the costs of the action. The question is one of considerable importance.

It is necessary to draw attention to the present unsatisfactory state of the law on the question as to what constitutes a "drain" and what a "sewer," and referred to in the Bill introduced into Parliament to amend the Public Health Acts Amendment Act, 1890. That Bill is necessitated owing to the difficulties created by s. 19 of this Act, as shown in a number of cases during the past few years, which led to strong expressions of opinion by the judges as to the desirability of an amendment to remove the anomalies caused by a strict interpretation of s. 19. Prior to that Act the definitions of "drain" and "sewer" were fairly clear. Thus s. 4 of the Public Health Act, 1875, defines these terms as follows: "'Drain' means any drain of and used for the drainage of one building only or premises within the same curtilage, and made merely for the purpose of communicating therefrom with a cesspool or . . . with a sewer into which the drainage of two or more buildings or premises occupied by different persons is conveyed, and 'sewer' includes sewers and drains of every description except drains to which the word 'drain,' interpreted as aforesaid, applies . . ." No practical difficulty in applying these definitions arose where houses were separately drained into a sewer, but the definition did not cover the case of a system of combined drainage of several houses, whether belonging to the same or different owners. To remedy this defect s. 19 of the Act of 1890 was passed, which contained the following provision—viz. that where two or more houses belonging to different owners are connected with a public sewer by a single private drain, an application may be made under s. 41 of the Public Health Act, 1875 (relating to complaints as to nuisances from drains), under which the expense of repairing drains may be thrown on the owners of such drains. It will be seen that this definition does not cover the case of a combined system where the houses belong to the same owner. This absurd result followed, that in the latter case the drain was a sewer, repairable, etc., by the local authority, while in the case provided for by s. 19 the drain was a drain, the repair of which might be thrown on the owner. This anomaly was revealed in the cases of *Thompson v. Eccles Corporation*, *Jackson v. Wimbledon Urban District Council*, and *Hardwicke v. Friern Barnet Urban District Council*, all reported in *Law Reports*, 2904, 2 K.B.; *Bradford v. Mayor, etc., of Eastbourne*, 1896, 2 Q.B. 205; and *Wood Green Urban Council v. Joseph*, 1907, 1 K.B. 187. To cure this anomaly is the object of Mr Vivian's Bill, which provides that a pair or group of houses owned by one person is to be treated in the same way as if owned by different persons—i.e. where they connect with a sewer by a private drain the latter will be treated in all cases as a drain repairable by the owner, and not as a sewer, within the meaning of s. 4 of the Public Health Act, 1875.

APPENDIX VII.

THE RANKINE-GORDON FORMULA

IN revising my book it occurred to me that the all-important formula known as the Rankine-Gordon, for use in designing pillars and stanchions, would still admit of a little more lucid explanation. Since writing, the following explanation has been placed before me by a professional colleague.

Let W be the load (originally axial) which is carried on a column with "hinged" ends and of unsupported length l . This length will not sensibly alter when bending takes place. The column is considered to bend practically into a circular arc; hence, by the properties of a circle,

$$x(2R - x) = \frac{l^2}{4} \quad \dots \dots \dots (a)$$

where

R = radius of curvature.

But it is proved in books on applied mechanics that if

e = modulus of elasticity,

y = distance of extreme fibre from neutral axis,

also M and p and I have the same meanings as they have had in the previous text, then

$$\frac{p}{y} = \frac{e}{R} = \frac{M}{I} \quad \dots \dots \dots (b)$$

Referring back to (a), since x is a very small amount, we may neglect x^2 , and so (a) becomes

$$x = \frac{l^2}{8R} \quad \dots \dots \dots (c)$$

and from (b)

$$\frac{l}{R} = \frac{p}{ey};$$

$$\therefore x = \frac{l^2 p}{8ey} \quad \dots \dots \dots (d)$$

$\frac{l^2}{8e}$ is an empirical constant for any particular column, and is usually denoted by c .

$$\therefore x = \frac{cl^2}{y}$$

M at the half-depth = Wx . Therefore stress due to bending alone

$$= \frac{Wxy}{I} \text{ (see } b)$$

$$= \frac{Wcl^2}{I}$$

Again, in books on mechanics it is shown that $I = AK^2$, where K is a term called "the radius of gyration," and a list of these terms can always be found therein.

$$\therefore p \text{ due to bending} = \frac{cWl^2}{AK^2}$$

If we denote $\frac{W}{A}$ by p_2 , we have stress due to bending

$$= cp_2 \frac{l^2}{K^2}$$

manufacturer who faces his blocks with materials which give them life is the one who gets the business.

Where a good, coarse sand is obtainable, it is sometimes more economical to use sand only in the proportions of one part of cement to four parts of sand, pit run. If the sand is coarse and well-graded, blocks made with these proportions will develop over 1000 lbs. per square inch in twenty-eight days.

It has always seemed to me that it is most important to get the requisite strength regardless of proportions; and if a one to four sand concrete will give these results, this should be permitted. The same holds true of a 1-3-6 mixture, provided it will give the required strength. Proportions of 1-2-4, 1-3-4, 1-2½-5, and 1-2-5 should all prove rich enough in cement to make blocks which will develop a crushing strength of 1000 lbs. per square inch in twenty-eight days, provided, of course, the aggregates used are of good quality and free from extraneous material.

CURING.

Wet tamped blocks should be sprinkled two or three times daily for a period of from fourteen to twenty-one days. They should be sprinkled after moulding as soon as the concrete has obtained its initial set, which it will do in from twelve to twenty-four hours.

The temperature of the curing-room should never be lower than the freezing-point of water, for if carried below this point the blocks will freeze, the setting of the cement will be delayed, and, even if the blocks are not injured, they will be no stronger when they thaw out than when they first froze, for the process of hardening or crystallisation stops at the time of freezing, and does not resume until the blocks are thawed out.

Many manufacturers supply blocks for use a few days after they are made. This is a great mistake and should never be done, as blocks cured by the sprinkled method for a few days are not strong enough to use in structural work, and it is errors of this kind that discredit the use of this material. Every now and then one can see blocks in buildings that are so soft and crumbly that the surface can be cut away, and this almost invariably is caused by insufficient water in the mixture or insufficient curing. It is sometimes caused by not using enough cement, but the former reasons are more often the cause.

WATERPROOFING.

This is a subject that has had the earnest attention of many people. As we have before stated, good, dense concrete, absolutely waterproof and impervious to moisture, can be made by the proper selection of aggregates, the proper mixing of the materials, the addition of a sufficient amount of water, and a proper curing of the stone. This is particularly true where a rich facing mixture is used for the block. To make assurance doubly sure, and to reduce the human equation to the lowest possible point, there have been devised waterproofing compounds and waterproofing fillers to make the blocks dense and impervious to moisture. Most of these compounds are water repellents. The use of them seems to me to be illogical, in view of the fact that the desired end in making concrete is to secure perfect crystallisation by the use of water.

APPENDIX IX.

METHOD OF CORRECTLY APPORTIONING THE COST OF REPAIRING A PRIVATE STREET.

BEFORE the surveyor proceeds to prepare a provisional apportionment under the Private Street Works Act, 1892, the authority must first resolve whether the apportionment shall be according to frontage, or if regard shall be had to the following considerations:—(a) The greater or less degree of benefit to any premises from such works. (b) The amount and value of any work already done by the owners or

occupiers of any such premises. They may also include any premises which do not so front, adjoin, or abut, but which have access from the street through a court, passage, or otherwise, and which, in their opinion, will be benefited by the works, and may fix the sum or proportion to be charged against any such premises accordingly. If the authority resolves to exercise their power under section 10, it will be necessary for the surveyor to take into consideration any special benefit which he may consider any of the premises derive from the works over and above other premises in the street. He must also take into account the amount and value of any work which may have been done. Under the 1875 Act the principle to be adopted in making the apportionment is based on frontage only. The preparation of an apportionment is a difficult work, and it is a matter of satisfaction that the tendency of the Court has been to give a somewhat liberal interpretation to the statutory powers possessed by local authorities in this matter, and not to attach too much weight to technical objections without any foundation in "merits." The provisional apportionment must show the amounts charged on the respective owners or reputed owners, also whether the apportionment is made according to the frontage of the respective premises or not, the measurements of the frontages, and the considerations (if any) on which the apportionment is based.

APPENDIX X.

EXAMINATIONS.

Most engineers like to belong to a representative body or institution of some sort; in fact, it is necessary to their welfare that they should be. Unfortunately engineers are not recognised by Government in the same way that doctors, barristers, clergymen, and lawyers are, and that such is the case is extremely detrimental to the profession at large. Large bodies of men are now wakening to this, and measures are being taken (without much success) to get the Government to take the matter in hand. At present anyone can call himself an engineer and put C.E. after his name; consequently the profession is overrun with such classes of men as plumbers, speculative builders, motor mechanics, electricians, etc., who, having got on in their respective trades, suddenly, without warning, become professional civil engineers. Such a fact is only too well known by the younger men who have had their articles in the proper way and can boast of a decent education, who, when they are out of their times, find the utmost difficulty in getting a position, and then, perhaps, only a poorly-paid and undignified one. To no class of men is less respect shown or more due than the municipal engineer, who has the all-important duty of public health at his control; and, strange to say, until quite recently this large body of disrespected public servants had no representative body at all. What steps should actually be taken to improve the profession it is very difficult to say: one thing at least is, however, essential, and that is, that no one shall be allowed to hold any appointment as, or call himself, an engineer till he has served a proper pupilage under a qualified man, and become a member of an engineering institution. But that is where the difficulty comes in—not so much the pupilage, but the institute. Foremost amongst the institutions comes the Institute of Civil Engineers, whose policy is very exclusive. This, however, is not the writer's business; suffice to say, it is so. The system of admission by examination is also a very debatable question. Many young college students can pass their exam. with ease, yet their practical experience may be nil (probably, so more often than not). Again, a prominent official desirous of entering probably has no time or inclination to study for academic examinations. On the other hand, to draw a dividing line as to who shall and who shall not be admitted is a very difficult task.

Regarding examinations, such must exist in cases, but if they are to do any good a drastic revision of present methods is essential. In the first place, there are too many frequently repeated "pet" questions of examiners.

Why are the excellent exams furnished by the City and Guilds Institute in London so little recognised? Surely honours in Mechanical Engineering is stiff enough for

anyone? There is much to be said for the *viva voce* method of examining. It is no doubt a practical test when administered by a thoroughly practical man.

The writer saw recently in *Surveying and the Civil Engineer* a very sensible remark by a correspondent on the matter of examinations, and repeats it in detail.

"Every engineering and architectural candidate should be sent to works in progress of construction, and be required to report and criticise in detail upon the design, construction, materials on site, and method of carrying out the work. Members of the professional institutions would readily give permission to examining bodies for candidates to visit works under their control for this purpose. This test would quickly reveal to any examiner what capabilities a candidate has of applying his theoretical knowledge to practical work."

Again, how many examination candidates retain the knowledge they acquired, in order to pass the exam, for twelve months afterwards?

However, at the present day examinations seem to hold a prominent place. All we can therefore say is, that young engineers must abide by this, and study for and pass all the examinations they possibly can, bearing in mind that youth is the time for such action, because when one gets older the desire for such study no doubt greatly diminishes. To aid the reader, therefore, to decide upon what to study, we give particulars and specimen papers of the more important examinations in connection with the profession.

Institute of Civil Engineers.

SYLLABUS OF THE EXAMINATIONS.

ADMISSION OF STUDENTS.

1. **ENGLISH** (one paper, *time allowed 3 hours*). A general paper comprising questions in Geography, History, and Literature.

2. **MATHEMATICS** (two papers, *time allowed 3 hours for each*). *Arithmetic*:—The principles and processes of Arithmetic applied to whole numbers and vulgar and decimal fractions; the metric system; approximations to a specified degree of accuracy; contracted methods of multiplication and division of decimals; ratio and proportion; percentage; averages. Practical applications of Arithmetic. *Algebra*:—Symbolical expression of general results in Arithmetic; algebraic laws and their applications; factors of simple binomial or quadratic expressions; equations of the first or second degree, and problems leading thereto; square root; graphs of simple rational integral algebraic functions; arithmetic and harmonic progression; geometric progression; theory of indices; logarithms; binomial theorem for a positive integral index. *Geometry*:—The subjects of Euclid I.-IV. with simple deductions, including easy loci and the areas of triangles and parallelograms of which the bases and altitudes are given commensurable lengths. (Euclid proofs will not be insisted upon.) *Trigonometry*:—Up to and including the solution of triangles, together with the practical solutions of triangles and applications, and numerical examples involving the use of logarithmic and other tables.

3. Two subjects, to be selected by the candidate from the following ten: a language is not compulsory, but in any case not more than one language may be taken (*time allowed 3 hours for each paper*):—

- (i) LATIN;
- (ii) GREEK;
- (iii) FRENCH;
- (iv) GERMAN;
- (v) ITALIAN;
- (vi) SPANISH;

Each paper will contain passages to be translated into English from books not previously prescribed, together with questions in grammar and a passage to be translated from English into the language.

- (vii) **ELEMENTARY MECHANICS** of solids and fluids.
- (viii) **ELEMENTARY PHYSICS**, including heat, light, electricity and magnetism.
- (ix) **ELEMENTARY CHEMISTRY**, including the principal properties of matter, the

laws of chemical affinity, atomic weights, valency, and the chemistry of the principal inorganic elements.

(x) **GEOMETRICAL AND FREEHAND DRAWING**, including projective and descriptive geometry and perspective.

ELECTION OF ASSOCIATE MEMBERS.

Part I.—General Knowledge.*

Candidates will be required to prove their knowledge of the English language and their attainments in general knowledge by writing an essay of about 1000 words upon a set subject. Each candidate will be allowed to select one out of several subjects of general knowledge, including English history and literature, prescribed by the Examiners (*time allowed 3 hours*).

Persons who are, or who have been, Students of the Institution, or who produce such qualifications as would, under the existing Regulations, exempt from the Examination applying to Admission of Students, will not be required to pass Part I. of this Examination.

Part II.†—Scientific Knowledge.

SECTION A.

1. **APPLIED MECHANICS** (one paper, *time allowed 3 hours*).
2. **STRENGTH AND ELASTICITY OF MATERIALS** (one paper, *time allowed 3 hours*).
3. *Either* (a) **THEORY OF STRUCTURES** (one paper, *time allowed 3 hours*).
or (b) **THEORY OF ELECTRICITY AND MAGNETISM** (one paper, *time allowed 3 hours*).

SECTION B.

Two of the following nine subjects—not more than one from any group (one paper in each subject taken, *time allowed 3 hours for each paper*):—

Group i.	Group ii.	Group iii.
GEODESY.	HYDRAULICS.	GEOLOGY AND MINERALOGY.
THEORY OF HEAT ENGINES.	THEORY OF MACHINES.	STABILITY AND RESISTANCE OF SHIPS.
THERMO- AND ELECTRO-CHEMISTRY.	METALLURGY.	APPLICATIONS OF ELECTRICITY.

Mathematics.—The standard of Mathematics required for the papers in Part II. of the Examination is that of the mathematical portion of the Examination for the Admission of Students, though questions may be set involving the use of Higher Mathematics.

The range of the examinations in the several subjects, in each of which a choice of questions will be allowed, is indicated generally hereunder:—

SECTION A.

1. **Applied Mechanics**:—

Statics.—Forces acting on a rigid body; moments of forces, composition and resolution of forces; couples; conditions of equilibrium, with application to loaded

* Candidates not exempt from Part I. must take that portion of the examination at the same time as Section A of Part II.

† Candidates may offer themselves for examination in Sections A and B of Part II. together; or they may enter for Section A alone, and, if successful, may take Section B at a subsequent examination. In the latter case, however, such candidates will not be allowed to present themselves for examination in Section B unless or until they are actually occupied in work as pupils or assistants to practising engineers. The Council may permit candidates who have attempted the whole of Part II. at one examination, and have failed in Section B only, to complete their qualification by passing in that section at a subsequent examination, subject to their being then occupied as above stated.

structures. The foregoing subjects to be treated both graphically and by aid of algebra and geometry.

Hydrostatics.—Pressure at any point in a gravitating liquid; centre of pressure on immersed plane areas; specific gravity.

Kinematics of Plane Motion.—Velocity and acceleration of a point; instantaneous centre of a moving body.

Kinetics of Plane Motion.—Force, mass, momentum, moment of momentum, work, energy, their relation and their measure; equations of motion of a particle; rectilinear motion under the action of gravity; falling bodies and motion on an inclined plane; motion in a circle; centres of mass and moments of inertia; rotation of a rigid body about a fixed axis; conservation of energy.

2. Strength and Elasticity of Materials :—

Coefficients of elasticity; elastic resistance to tension, compression, shearing and torsion; uniform and varying stress; moment of stress and of resistance in beams of various sections; distribution of shearing stress in beams; stresses suddenly applied and effects of impact; buckling of struts; effect of different end-fastenings on their resistance; combined strains; calculations connected with statically indeterminate problems, as beams supported at three points, etc.; limit of elasticity, yield-point, and ultimate resistance of various materials as tested; the plastic state; principal forms of testing-machines and of appliances used in measurements for the determination of coefficients of elasticity; calculation of extension, deflection, buckling, etc., within elastic limits, and of ultimate strength and ductility for ordinary materials of construction.

3. (a) Theory of Structures :—

Graphic and analytic methods for the calculation of bending moments and of shearing forces, and of the stresses in individual members of framework structures loaded at the joints; plate and box girders; incomplete and redundant frames; theory of continuous girders and principal methods of calculation; travelling loads; riveted and pin-joint girders; rigid and hinged arches; strains due to weight of structures; theory of earth-pressure and of foundations; stability of masonry and brickwork structures.

3. (b) Theory of Electricity and Magnetism :—

Electrical and magnetic laws, units, standards, and measurements; electrical and magnetic measuring instruments; the theory of the generation, storage, transformation and distribution of electrical energy; continuous and alternating currents; arc and incandescent lamps; secondary cells.

SECTION B.

Group i. :—

Geodesy :—

The theory, structure and adjustment of the principal surveying and levelling instruments, and the principles of their employment under various conditions; errors of observation and their elimination; determination of position and time; plane and geodetic triangulation; land surveying; contouring; setting out curves; mining surveying and hydrographical surveying; observation of currents and of tides, with determination of mean sea-level.

Theory of Heat-Engines :—

Thermodynamic laws; internal and external work; graphical representation of changes in the condition of a fluid; theory of heat-engines working with a perfect gas; air- and gas-engine cycles; reversibility, conditions necessary for maximum possible efficiency in any cycle; properties of steam; the Carnot and Rankine (or Clausius) cycles; entropy and entropy-temperature diagrams, and their application in the study of heat-engines; actual heat-engine cycles and their thermodynamic losses; effects of clearance and throttling; initial condensation; testing of heat-engines, and the apparatus employed; performances of typical engines of different classes; efficiency.

Thermo- and Electro-Chemistry :—

The nature of the principal fuels; theory of combustion; calorific value of carbon and hydrogen as used for the production of heat, and of the principal fuels—solid, liquid and gaseous; examination of the chemical processes which occur in the working of open and closed furnaces, and in the working of engines either by coal or producer-gas, or by petroleum, etc.; theory and use of the principal apparatus employed in the determination of calorific values; methods of production and calorific value of coal-gas, water-gas, and producer-gas; calculation of thermal changes in industrial processes; calorific intensity, and the instruments used for its measurement; theory of electrolysis, and of the principal electrolytic processes; chemistry of primary and secondary batteries; the use of electric currents for heating and for chemical decomposition.

Group ii. :—

Hydraulics :—

The laws of the flow of water by orifices, notches and weirs; laws of fluid friction; steady flow in pipes or channels of uniform section; resistance of valves and bends; general phenomena of flow in rivers; methods of determining the discharge of streams; tidal action; generation and effect of waves; impulse and reaction of jets of water; transmission of energy by fluids; principles of machines acting by the weight, pressure and kinetic energy of water; theory and structure of turbines and pumps.

Theory of Machines :—

Kinematics of machines; inversion of kinematic chains; virtual centres; belt, rope, chain, toothed and screw gearing; velocity, acceleration and effort diagrams; inertia of reciprocating parts; elementary cases of balancing-governors and fly-wheels; friction and efficiency; strength and proportions of machine parts in simple cases.

Metallurgy :—

The physical properties of metals generally used by engineers; structure, density, thermal and electric conductivity; microscopic examination of metals; modifications produced by local stresses, cold rolling, hard drawing and hammering; nature of annealing.

The production of high temperatures; pyrometry; the chemical reactions that occur at high temperatures, so far as they affect metallurgical processes; refractory materials; furnaces and combustion-chambers; fuels; principles of iron and steel manufacture by modern methods; the influence of carbon and other substances upon iron and its alloys; the principal ores and processes of extraction of copper, lead, tin, zinc, silver and gold, and the composition and properties of their chief alloys; the laws of electrolysis as regards their application to metallurgical processes; electro-heating for metallurgical purposes.

Group iii. :—

Geology and Mineralogy :—

The nature of common minerals and the methods by which they can be discriminated; mode of occurrence of minerals in rock and in mineral-veins; composition and classification of the rocks forming the earth's crust; changes taking place in the earth's crust by the action of the atmosphere and of water upon it; denudation and the formation of fluvial, glacial, lacustrine, deltaic, and marine deposits; action of internal forces upon the globe; volcanic phenomena and earthquakes; movements of terrestrial rock-masses; methods of measuring and recording in sections and maps the positions of the rock-masses of the globe; the general succession of the stratified deposits in the British Isles, based on the study of the fossils which they contain.

Stability and Resistance of Ships :—

The principles of buoyancy, stability and oscillation for floating bodies, with special application to ships, and methods of calculation and experiment; principles of the structural strength of ships; the straining forces acting upon ships afloat and

- [25] 6. Derive the formula for the torsional resisting moment of a solid cylindrical shaft. What will be the maximum fibre stress of a 3-in. shaft transmitting 100 H.P. at 300 revolutions per minute?
- [20] 7. Rankine's formula for the crippling load P of a round-ended column of cross-sectional area A is

$$\frac{P}{A} = \frac{f}{1 + a \frac{l^2}{k^2}}$$

where l denotes the length of the column and k the radius of gyration of the section, also $a = \frac{1}{7600}$ and $f = 48,000$ lbs. per sq. in. for mild steel.

Use this formula to find the crippling load of a round-ended column 15 ft. long made of a 10 × 8-in. rolled steel joist whose least moment of inertia is 71.6 in. units, and whose area is 20.6 sq. in.

- [20] 8. A cylindrical steel tank, 30 ft. in diameter, is to be filled with water to a depth of 12 ft. What is the thickness of plate and what riveting in the vertical joints would you adopt for the bottom ring of plate?

(Weight of water = 62½ lbs. per cub. ft.)

- [20] 9. What is the least internal radius to which a flat bar $\frac{1}{2}$ in. thick can be bent so that the maximum stress does not exceed 7 tons per sq. in., taking the modulus of elasticity at 14,000 tons per sq. in.?
- [25] 10. Derive from first principles the moment of resistance of a rectangular beam.

(Breadth = b , depth = d .)

- [20] 11. Describe the effects produced by the several processes of hardening, tempering, and annealing upon the physical properties of steel containing various percentages of carbon, from the point of view of the use of the material for engineering purposes.
- [20] 12. State the physical conditions and phenomena to be determined or observed in the practical examination of Portland cement, and the methods of testing it, giving an outline sketch of some common form of machine for determining its tensile strength.

3. (a) THEORY OF STRUCTURES.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [20] 1. Describe the "Method of Sections," and by this principle ascertain the stresses in the bars AB, AC, BC of the frame shown in fig. 1 [= 1063a]:—

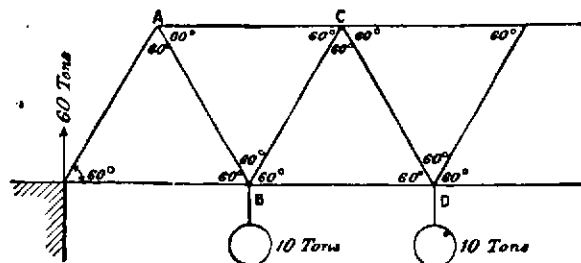


FIG. 1063a.

- [20] 2. A beam 25 ft. long is supported at one end and at a point 5 ft. from the other end; it is loaded with a distributed load of 10 cwt. per ft.-run, and has a load of 5 tons suspended from the end of the overhanging portion of the beam. What is the pressure on the supports, and the amount and position of the maximum bending moment? Draw a diagram of the bending moments and shearing forces.

3. A strut 30 ft. long is constructed of two channel bars 15 in. × 4 in., braced together with diagonal angle bars 3 in. × 3 in. × $\frac{1}{2}$ in. as shown in fig. 2 [= 1063b]. Find the distance marked x , so that the ratio $\frac{l}{r}$ of this length unsupported between the bracing bars does not exceed 60 per cent. of the $\frac{l}{r}$ of the whole column.

The maximum moment of inertia of the channel bar is 377 in. units and the minimum is 14. The centre of gravity is 1 in. from back of channel. Area of channel = 12 sq. in., l = length of column, r = radius of gyration.

4. Sketch and dimension two types of movable bearings for the ends of straight girder bridges, assuming the load on the bearing to be 50 tons.

5. Sketch and dimension the joint of boom and bracing bars of a braced girder bridge, as shown in fig. 3 [= 1063c]. The top boom consists of two channel bars 15 in. × 4 in. × $\frac{1}{2}$ in. placed 9 in. apart back to back; the verticals consist of two channel bars 10 in. × 4 in. × $\frac{1}{2}$ in., and the diagonal tension bars consist of two flat bars 7 in. × $\frac{1}{2}$ in.

6. A strut or column 10 ft. long consists of two T bars 5 in. × 2½ in. × $\frac{1}{2}$ in. riveted together back to back; adopting the formula $P = 6 - \frac{1}{40} \frac{l}{r}$ as the allowable stress per sq. in. on the column, what load will the column stand so as not to exceed this stress? Assume the corners of the T bars to be rectangular and the sides parallel. What do you consider the maximum $\frac{l}{r}$ ratio allowable? l = length of column, r = radius of gyration.

7. Compare the respective advantages and disadvantages of the two types of bascule bridges known as the "Trunnion" and the "Scherzer rolling lift bascule."

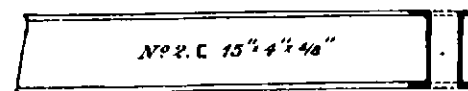


FIG. 1063c.

8. Describe two methods of stiffening the deck of a suspension bridge from vertical oscillations, and state the advantages and disadvantages of each.

9. The central portion of a cantilever bridge is being lifted into position, as shown in fig. 4 [= 1063d]. Neglecting inertia and friction, what is the stress in the member marked A, the suspended load on the tackle being 60 tons?

10. The dock gates of a 60-ft. entrance are segmental in plan, having a versed sine of 6 ft. With a depth of water of 20 ft., what is the pressure on a strip 12 in. wide vertically on the hollow quoins and at the jointing of the gates, the centre of the strip to be at a distance of 5 ft. from the bottom? Assume weight of water to be 64 lbs. per cubic ft.

11. Give a résumé of the results of Wöhler's or similar experiments on repeated application of a load, and state how these results affect the designing of girder bridges.

12. A concrete dam is triangular in section and is built of stone, having a density

ashore; methods of calculation for strength and stiffness; nature of the wave-forming and other resistances experienced by ships moving through water; stream lines; methods of model experiments on resistance; influence of forms and proportions upon resistance; mechanical theory of propellers and relative efficiency of various kinds of propellers; estimates of horse-power for assigned speeds; manœuvring of ships under the action of rudders or propellers.

Applications of Electricity:—

Theory and design of continuous- and alternating-current generators and motors, synchronous and induction motors and static transformers; design of generating- and sub-stations, and the principal plant required in them; the principal systems of distributing electrical energy, including the arrangement of mains and feeders; estimation of losses and of efficiency; principal systems of electric traction; construction and efficiency of the principal types of electric lamps.

PART I.—GENERAL KNOWLEDGE.

(Maximum marks 100.)

Write an Essay on one of the following subjects:—

- Ship Canals.
- The relative advantages of the Workshop and the Engineering School of a University, as a training for the Engineer.
- The bearing of the Study of Science upon Character.
- The relative advantages for town life of tenement-dwellings and the separate-house system.

PART II.—SCIENTIFIC KNOWLEDGE.

Section A.

1. MECHANICS.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [20] 1. What are the relations between the pound, the poundal, and the dyne? Under what circumstances may the pound be used as a unit of force without sensible error? Mention some other cases in which its use would be quite inadmissible.
- [20] 2. Two weights, A and B, are suspended from the two ends of a light silken cord which passes over a frictionless pulley. By the accelerated descent of the greater weight, A, through a fall of 8 ft., the smaller weight, B, is raised through the same height in 2 seconds of time (starting from a condition of rest). The weight, B, is 1 lb.: how much is the weight A? Neglect the mass of the cord and of the pulley.
- [25] 3. A horizontal girder 60 ft. in length, and carrying a uniform load of 2 tons per ft., is lifted by three hydraulic cylinders, which are applied one under each end and one under the centre of the girder. The three cylinders are all of the same diameter and in hydraulic communication with each other. Find the bending moment at the centre of the girder. Determine also the greatest bending moment and the point where it takes effect upon the girder.
- [20] 4. A load of 2 tons is suspended by a vertical rope 300 ft. long, the rope itself weighing 6 lbs. per ft. In winding up the load to the top, how many ft.-lb. of work are done?
- [20] 5. A cage, weighing with its load 5 tons, is lifted by a winding-engine at the maximum working speed of 30 ft. per second. The maximum speed is attained by a uniform acceleration in a period of 6 seconds after starting. Find the tension in the wire rope during this period of time.
- [20] 6. In the vertical face of a dam, a circular opening 4 ft. in diameter is closed by

a sluice-gate, the water standing 2 ft. above the upper edge of the opening. Find the total hydrostatic pressure upon the gate, and the exact height of its centre of pressure.

7. A simple triangular roof-truss, A B C, consists of a horizontal tie-beam, B C, [20] 10 ft. long, supported at each end, and two inclined rafters, A B and A C, which are respectively 6 ft. and 8 ft. in length, meeting at the ridge A. Determine the stress in each of the three members due to a load of 1 ton imposed upon the ridge A.

8. A straight horizontal beam, A B C D, whose length, A D, is 100 ft., and its [20] weight 50 lbs. per lineal ft., is supported and held down to an abutment at A, and supported also at C, 40 ft. from A (without being fixed in direction). Find the external forces or reactions at A and C due to the weight of the beam; and also the value of those forces when the beam carries a load of 2000 lbs. at B, which is 10 ft. from A.

9. While a railway train is running at 40 miles an hour upon a falling gradient [20] of 1 in 100 (without steam), the brakes are put on, applying a total retarding force which is equivalent to one-twentieth of the weight of the train. In what distance and in what space of time will the train be stopped?

10. A cubical box, 2 ft. by 2 ft. by 2 ft., partly filled with water, is made to rotate [25] about a vertical axis passing through the centre of its base, at a uniform speed of 60 revolutions per minute. At the centre the water stands 3 in. above the floor of the box. To what height does it rise at each of the four corners? Trace the curve of the water-surface upon either of the four vertical walls, writing its equation.

11. A double-armed swing-bridge revolves upon a horizontal turn-table at the [25] centre of its length, being actuated by a chain-drum 32 ft. in diameter. The two main girders of the bridge, 180 ft. in length, are spaced 30 ft. apart, transversely (centre to centre), and each girder has the uniform weight of 10 cwt. per ft. Reduce the inertia of these revolving girder-masses to the driving point.

12. Write the equations for the two curves which are described by the diametral [25] section of a forced vortex and of a free vortex respectively.

2. STRENGTH AND ELASTICITY OF MATERIALS.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

1. A timber beam of 12 ft. span is supported at both ends; it is 8 in. deep and [20] 14 in. wide, and carries a brick wall 14 in. thick. Find what height of wall it can carry if the maximum stress in the beam is to be 800 lbs. per sq. in., and the weight of brickwork is 125 lbs. per cub. ft.
2. Draw to scale the stress-strain diagram of a tensile test of a steel bar 1 in. [20] square, and having an ultimate strength of 30 tons per sq. in. Explain the meaning of the different parts of the diagram.
3. A load of 3 tons is hung from a round steel rod $\frac{3}{4}$ in. in diameter and 40 ft. [20] long. How much will the rod stretch if its modulus of elasticity be 13,000 tons per sq. in.?
4. A rolled steel joist, 8 in. deep, whose moment of inertia is 55.7 (in inch units), [25] is laid on bearings 10 ft. apart, and carries a load of 7 tons uniformly distributed. What is its maximum fibre stress, and what is its central deflection if the modulus of elasticity be 14,000 tons per sq. in.?
5. In a certain girder a steel flat bar tension member, 10 in. wide and $\frac{1}{2}$ in. thick, [25] carries a load of 30 tons; it is attached to a gusset by a butt joint with two cover-plates, rivets of $\frac{3}{4}$ in. diameter being used. Give a dimensioned sketch of the riveting you would propose, and explain fully your reasons for the number of rivets, pitch, lap, and thickness of cover-plate which you adopt.

Assume the following as limiting stresses:—

5.5 tons per sq. in.	in shearing for rivets.
11.0 " "	bearing " plates.
6.5 " "	tension " "

2.24 compared with water. What must be the thickness of the dam at a depth of 30 ft. if the line of resultant pressure is to cut the section at a distance of two-thirds of its breadth from the vertical inner face, with a full head of water? What is the pressure on the toe of the dam on opposite side of reservoir, when the dam is empty, assuming the pressure on the bed to be uniformly varying?

3. (b) THEORY OF ELECTRICITY AND MAGNETISM.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [20] 1. Calculate the volts between the ends of a piece of copper wire one kilometre

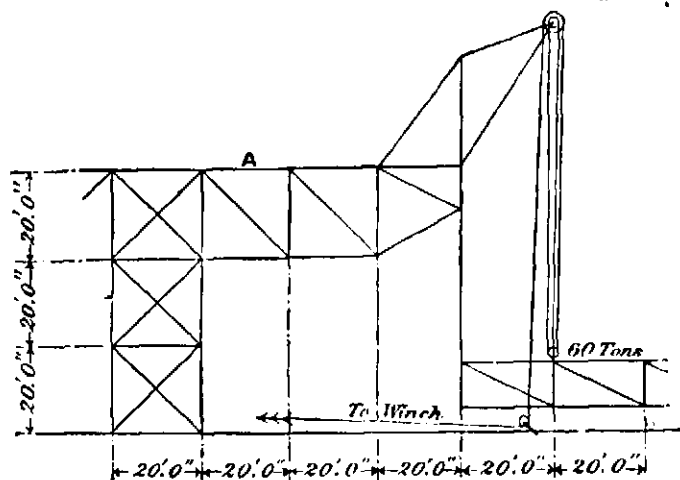


FIG. 1063d.

long, 0.914 mm. diameter, at a temperature of 40° C. when a current of 1.8 amperes is passing.

Specific resistance at 15.5° C. = 1.73×10^{-8} ohms.

Temperature coefficient = 0.00428 per degree C.

- [20] 2. Give the Board of Trade definition of the unit of electrical measurement, the ampere, and state its value in terms of the centimetre, gramme, and second.
- [20] 3. Sketch the essential parts of, and describe one good form of, portable ohmmeter. State the principles involved in its working.
- [20] 4. Prove that the power in a three-phase system can be measured by means of two wattmeters, and give a diagram showing the way in which the instruments would be connected to the system.
- [25] 5. A storage battery is worked in connection with a variable load and is intended to keep the load on the station approximately constant. Give a diagram of the connection, and show how this can be automatically carried out by aid of a suitably designed booster.
- [20] 6. Explain the advantages of the three-wire system of direct current distribution, and give a diagram showing how dynamos, instruments, and lamps would be arranged in connection with it.
- [20] 7. Show how you would obtain the mean spherical candle-power of an arc-lamp, having given the polar-curve of candle-power.
- [20] 8. What is the relation between the torque exerted by a direct current motor and

the other quantities upon which it depends? Give a diagram showing how the torque, efficiency, and speed of a direct current series motor vary in terms of the current supplied to it at fixed voltage.

9. In connection with an inductive circuit, explain the meaning of the terms [25] "reactance," "inductance," and "impedance." How and why would the above quantities be affected by variation of frequency in the case of an iron rod when traversed by an alternating current?

10. A circuit consists of an inductive air coil in series with a condenser, and is [25] connected to a source of alternating currents. Explain why the potential difference across the condenser may be greater than that across the alternating supply system. With a given coil, impressed voltage, and frequency, what value of the capacity will give maximum voltage across the condenser?

11. In connection with dielectrics, explain the meaning of the terms "specific [20] inductive capacity," "dielectric strength," and "specific resistance." How are these quantities affected by variation of temperature?

12. Why is it that, for a given length of line, the three-phase system is more [25] economical in copper than a single-phase system? Prove what the economy amounts to if the efficiency is to remain constant.

Section B.

Group i.—GEODESY.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

1. The latitude of B from A is 106.42 ft. N. and its departure is 273.62 ft. W. [20] What is the bearing of the line AB?
2. You have to make a longitudinal section along a centre line which has been [20] pegged out in the field. What instruments, equipment, and labour would you take into the field with you?
3. How would you determine the instrument constant and focal correction of a [30] telescope fitted with stadia lines?
4. Give two methods of finding your position in a survey by means of the [20] plane table. Of the two methods selected which do you consider the better, and why?
5. Describe the permanent adjustments of a dumpy level. Explain particularly [20] the effect of moving the various screws used in the adjustments.
6. A prismatic compass traverse closes on its starting-point. In plotting the field, [20] observations you find there is a closing error. How would you eliminate it? (The bearings have been corrected for local attraction before plotting.)
7. You are given a 6-inch Ordnance map with contours of 100 ft. vertical interval. [20] How would you interpolate contours at 25 ft. interval, using a clinometer?
8. Two lines meeting at an angle of 80° are to be joined by a 4° curve. A, the [25] starting-point of the curve, is at chainage 10,560 ft. You are pegging out the curve at 100-ft. intervals. Setting up the theodolite at A, you peg out the curve to chainage 10,800 ft., and then find that an obstacle prevents you from seeing any further. Explain how you would peg out the remainder of the curve.
9. Explain the field observations necessary for finding the latitude by circum- [25] meridian altitudes of the sun. Note generally the corrections and balancing of the observations necessary for reducing the observed altitudes to meridian altitudes. (Formulas are not required.)
10. Explain what is meant by geocentric latitude and geographical latitude. Show [25] how local attraction affects the determination of the latitude.
11. A base for a topographical survey is measured with a steel band. Explain [25] the corrections applied to the field measurements in order to reduce the base for computation of the triangulation.
12. Explain how the direction of a survey line above ground is transferred to the [20] bottom of a shaft.

Group i.—THEORY OF HEAT-ENGINES.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [20] 1. 5 lbs. of air at 250 lbs. pressure and 285° F. is allowed to expand to atmospheric pressure in a cylinder of which the sides are made non-conducting. Find the final temperature and volume and external work done.
- [25] 2. Air is compressed in a two-stage compressor, the pressure at the end of the first stage being 5 atmospheres, and at the end of the second 200 atmospheres. Assuming that the compressions are adiabatic and that 20 cubic ft. of air at atmospheric pressure and 60° F. temperature are supplied per minute, find the approximate amount of cooling water supplied at 60° F. per minute required to keep the average temperatures in the compressor constant, and calculate the horse-power required to drive the machine if its mechanical efficiency is 75 per cent.
- [20] 3. Show that for a perfect gas the difference between the specific heats at constant pressure and constant volume is equal to the work done in raising the gas one degree in temperature. Hence show that if the absolute temperature and pressure of a gas are known its volume is also known.
- [25] 4. Saturated steam at 100 lbs. atmospheric pressure ($t = 327.6^\circ \text{ F.}$) is allowed to expand from one vessel into another, increasing its volume fivefold. Assume the vessels to have non-conducting walls and that the steam was initially dry, find the percentage of moisture at the end of the process. The volume of the water may be neglected. The specific volume of saturated steam is given by the equation

$$\log v = 2.516 - 0.039 \log p.$$

$$\text{Latent heat of evaporation} = 1092 - 0.7(t - 32).$$

$$\text{Lower temperature} = 225^\circ \text{ F.}$$

- [20] 5. Show how the entropy temperature diagram for a steam engine may be used to investigate the amount of moisture present at various points in the revolution.
- [20] 6. Sketch the entropy temperature diagram for (a) a gas engine of any well-known type, (b) a Diesel oil engine.
- [25] 7. A steam-engine cylinder is 12 in. diameter and 3 ft. stroke, and the engine runs at 100 revolutions per minute. The steam pressure is 150 lbs. absolute, and the cut off at $\frac{1}{2}$ stroke. Sketch in cross section, and give the leading dimensions of the steam and exhaust ports and slide valve, stating also the throw of the eccentric.
- [20] 8. Show by the help of a Zeuner diagram the effect on the steam distribution of linking up a Stephenson link valve motion.
- [20] 9. Sketch in section either (a) a boiler of the Lancashire type, or (b) a water tube boiler of the express or small tube type, or (c) a gas producer of the suction type, and show in detail the method of staying the boiler chosen, or the method of cleansing the gas as it passes from the producer.
- [25] 10. A gas engine of 500 B.H.P. consumes 16 cubic ft. of coal gas per B.H.P. per hour. The difference between the specific heats of the gas at constant pressure and constant volume is 120 B.Th.U. and of the mixture of air and gas after explosion 56. The ratio of air to gas by volume is 10 to 1. The pressure at the end of the suction stroke is 13 lbs. absolute, and the temperature of air and gas at the end of the suction stroke is 100° F., the pressure at the end of compression 60 lbs. per sq. in. absolute, at the end of explosion 144 lbs. per sq. in. absolute, and temperature 2500° F., and constant pressure expansion takes place at that pressure until a temperature of 2700° is reached, expansion then following the law $PV^{1.33}$ constant. If the ratio of expansion is 4 to 1, find the clearance volume, temperature at end of compression, weight of the mixture, heat units added during explosion (at constant volume), and temperature of release assumed to be at the end of the stroke and heat rejected in the exhaust.
- [20] 11. Give a diagrammatic sketch of the arrangement of (a) a Parsons turbine, showing the balancing arrangement for endlong thrust; (b) a Curtis turbine with shape and arrangement of nozzles; (c) a de Laval turbine with nozzles in position. State the type of work which each is most suited for.

12. How is a petrol engine rated for power? Show how the actual power developed compares at, say, three speeds with the rating. [20]

Group i.—THERMO- AND ELECTRO-CHEMISTRY.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

1. The law of maximum heat evolution may be stated thus:—"Chemical change which is effected without the addition of external energy happens of necessity if there is an evolution of heat during the change." Give examples of the application of this law and discuss its limitations. [25]
2. What do you understand by the latent heat of steam? Describe and explain a method of measuring it. [20]
3. When potassium nitrate is dissolved in water heat is absorbed, but the reverse is the case when calcium sulphate is dissolved. Discuss the effect of rise of temperature on the solubility of these substances. [20]
4. Explain the meaning of the terms "critical temperature" and "critical pressure" as applied to gases and vapours. Describe an experiment in illustration of your answer. [20]
5. Give a general account of the construction of an "explosion bomb" and describe its use in the determination of the calorific value of a sample of coal. [25]
6. What do you understand by the statement that the specific heat of iron is 0.114? Describe and explain the method you would adopt for determining this value. [20]
7. Give an outline of the present theory of the passage of a current through an aqueous solution of an electrolyte, stating briefly the chief facts on which it is based. [20]
8. Describe the construction of a Leclanché cell. State the function of each part of it and explain the chemical reactions involved in its use. [20]
9. The molecular conductivity of a solution of a salt, e.g. silver nitrate, increases with dilution. What do you understand by this statement, and what explanation of the fact has been offered? [20]
10. What do you understand by polarisable and non-polarisable electrodes? Give examples and describe methods of preventing polarisation in a cell. [25]
11. A current of electricity is passed separately through (a) fused sodium chloride, (b) through an aqueous solution of sodium chloride, (c) through dilute sulphuric acid. Describe the changes in each case, and state what substances separate at the electrodes. Assume that the latter are not chemically affected. [20]
12. What experiments would you make to ascertain the electrical relations of zinc and iron and of tin and iron to one another, discussing the conditions of the experiments you describe and the deductions you would make from them? [25]

Group ii.—HYDRAULICS.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

1. Twelve hundred gallons of water per minute are to be lifted a height of 150 ft. through a pipe 1 ft. diameter and 5000 ft. long. Assuming the head lost by friction in the pipe per ft. length when the velocity is one ft. per second is 0.0005 ft., and the mechanical efficiency of the pump 80 per cent., find the effective horse-power of the engine driving the pump. [20]
2. The outlet circumference of a centrifugal pump has a diameter of 2 ft. and the inlet circumference is 1 ft. 3 in. diameter. The radial velocity for flow at inlet is 5 ft. and at outlet 8 ft. per second. The wheel makes 400 revolutions per minute. The vanes are radial at exit. Determine the theoretical lift of the pump and the direction of the inlet edge of the vane. [25]
3. Show that Bernoulli's theorem is consistent with the principle of the conservation of energy. One cub. ft. of water per second flows along a pipe, which has a slope of 1 in 10, from A to B. At A the diameter is 3 in. and at B 6 in., and [20]

the horizontal distance from A to B is 50 ft. The pressure head at A is 20 ft. of water and at B 25 ft. Find from this data the head lost between A and B.

- [20] 4. A turbine is driven by a head of 120 ft. The water used in a test of the turbine is measured by passing it through a right-angled V notch. The mean head over the apex of the notch during the trial is found to be 9 in. The brake horsepower of the turbine is found to be 13; find the efficiency of the turbine.

- [25] 5. A horizontal pipe A C, fig. 1 [= 1063c], has a diameter of 6 in. and is 1000 ft. long. At point B a horizontal branch-pipe 3 in. diameter and 500 ft. long is taken off A C. The pressure of A, C, and D are found to be 20 ft., 10 ft., and 7.5 ft. of water respectively. Assuming the head lost by friction in a length l of either of the pipes to be

$$\frac{0.0006 l v^2}{d}$$

and neglecting other losses, show that the velocity in A B is about 3.16 ft. per second and find the pressure at B.

- [20] 6. Water issues from a square-edged vertical circular orifice 2 in. diameter, made in the side of a vessel in which a constant pressure of 200 lbs. per sq. in. is maintained, and the head of water in the vessel above the centre of the orifice is 8 ft. A flat plate is placed near to the orifice, the plate being inclined at an angle

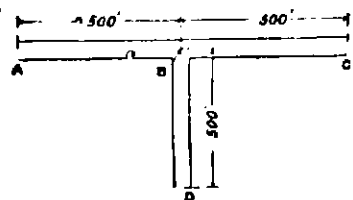


FIG. 1063c.

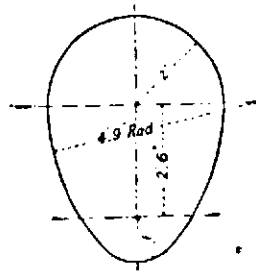


FIG. 1063f.

of 60° to the horizontal plane. Find the discharge from the orifice and the horizontal pressure on the plate.

- [20] 7. Describe briefly, with the aid of sketches, two methods of determining the discharge of a river 100 ft. wide.

- [20] 8. Make sketches of and explain the principle of action of one of the following:—

- (a) A compressed air pump showing the lift, and the depth of water in the well.
(b) A hydraulic ram.
(c) A hydraulic press in which the ram is returned automatically.

- [25] 9. Make sketches of two arrangements for regulating the flow of water to pressure turbines and of one arrangement for regulating the flow to one form of impulse turbine. Discuss the effects of the arrangements in diminishing the efficiency of the turbines.

- [25] 10. A hydraulic direct-acting lift has a ram 9 in. diameter. The pipe connecting the valve box to the cylinder is $\frac{1}{2}$ -in. diameter. The pressure in the valve box is 600 lbs. per sq. in. Neglecting all other losses than that due to sudden enlargement when the water enters the cylinder, and assuming the valve fully open at all loads, find the maximum load that can be lifted at a velocity of 1 ft. per second. Find also the maximum velocity with which the lift could descend, assuming the exhaust full open.

- [20] 11. A brick-lined sewer has a section shown in fig. 2 [= 1063f], and a slope of 1 in 1000. Determine the discharge of the sewer in cub. ft. per second when the depth of flow is 2 ft. Explain the formula you use.

Note:—Areas, etc., may be found from a scale drawing.

- [25] 12. Give an account of any experiments with which you are familiar on the

discharge of water over weirs. State the deductions to which the experiments would lead you in making a weir for the accurate gauging of water, with respect to the following points:—

- (a) the form of the sill of the weir.
(b) the form of the approaching channel.
(c) any precautions to be taken in connection with the downstream channel.

Group ii.—THEORY OF MACHINES.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each is shown in brackets.

1. The connecting-rod of a steam engine is 52 in. long, and the crank radius is 13 in. Find the angular velocity of the crank shaft relative to the main bearing, the angular velocity to the crank-pin relative to the connecting-rod bearing, and the angular velocity of the connecting-rod relative to the crosshead pin, when the crank has turned through 30° from the inner dead centre and the revolutions are 200 per minute. The angular velocities are to be expressed in radians per second. [20]

2. The reciprocating parts of a steam engine weigh 8½ tons; the connecting-rod is 17.5 ft. long, and the crank radius is 3 ft. Calculate the force required for acceleration of the reciprocating masses at the beginning and end of the stroke, respectively, when the speed is 60 revolutions per minute. [20]

3. A train weighing 200 tons is travelling at 40 miles per hour up a bank of 1 in 75, against a tractive resistance of 25 lbs. per ton. Find the horse-power necessary to maintain the speed. [20]

4. The tup of a drop forging plant is sometimes lifted by means of a belt passed over a revolving pulley on a main shaft running above the hammers. The belt over the pulley is arranged so that one end is attached to the tup whilst the other end hangs free, and within reach of the smith operating the hammer. [20]

Suppose the arc of contact between the belt and the pulley is 180°, and that the coefficient of friction between the leather and the pulley is 0.5, find what weight of tup could be lifted when the smith exerts a pull of 50 lbs. on the free end of the belt.

5. The cylinders of a four-cylinder vertical marine engine are pitched at equal distances apart. The reciprocating masses belonging to each of the two middle lines of parts weigh 1 ton. The cranks of the middle pair of cylinders are set at an angle of 100°. Find the angle between each outside crank and its neighbour, and the weight of each of the outer reciprocating line of parts so that the engine shall be in balance for primary forces and couples. [20]

6. Make a sketch of any one form of governor in which the control is effected mainly by a spring. [20]

Distinguish between stable and unstable conditions of running, and show by the aid of a diagram how the stability is affected by the initial compression (or tension) of the controlling spring.

7. Make a sketch of the application of an epicyclic train of wheels to any purpose which has come under your notice, and show how to find the relative velocity of the first and last piece in the train. [20]

8. The difference between the driving and the back pressures in a steam-engine cylinder is 60 tons when the crank is 30° from the inner dead point. The reciprocating parts weigh 7 tons. The connecting-rod is 5 times as long as the crank, which is 3 ft. radius. Calculate the corresponding value of the turning moment in the crank shaft when the speed is 60 revolutions per minute. (Any graphical methods may be used in the solution of this problem.) [25]

9. Make a sketch of a piston valve in a central position over the ports, and arranged to take steam on the inside of the valve. [25]

Assuming the valve to be operated by a simple valve gear with a single eccentric sheave, find the radius of the eccentric sheave, its angular position with regard to the crank, and the steam lap, so that cut off may take place when the crank is at 120° from the dead point, the lead and the maximum port opening for steam being respectively $\frac{1}{16}$ in. and 1 in. (Neglect the effect of the obliquity of the connecting-rod.)

- [20] 9. Indicate on the outline sheer plan of a vessel the parts where maximum stresses on shell-plating and rivets are likely to be experienced when the ship is in a seaway. Give reasons for the statements made and say what would be the nature of the stresses.
- [25] 10. Give a sketch of the wave systems which accompany a vessel in her motion through the water. Certain of these waves "interfere" with others; point out the effect of this upon the resistance of the ship at different speeds.
- [25] 11. The resistance of a model when towed at 4 knots speed is 7.08 lbs., of which 4.8 lbs. is due to surface friction. Make an estimate of the I.H.P. required to drive at 20 knots a ship having all its dimensions twenty-five times those of model, the wetted surface of vessel being 21,800 sq. ft. and its frictional resistance per sq. ft. at a speed of 1 knot being 0.00889 lb.
- [25] 12. A screw propeller 32 in. in diameter delivers a thrust of 192 lbs. when working at 300 revolutions per minute with a certain slip ratio. What thrust will be delivered by a screw of exactly similar design 80 in. in diameter when working at 600 revolutions per minute with the same slip-ratio as smaller screw?

Group iii.—APPLICATIONS OF ELECTRICITY.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [25] 1. Discuss the effect of using a battery on the efficiency of a traction substation which has a very peaky load. Would the battery have any other advantages?
- [20] 2. How long do you estimate it would take for a current of 5 amperes at 100 volts to boil a tea-kettle holding 1 quart of water?
- [20] 3. What is the maximum drop in voltage allowed by the Board of Trade in tramway rails? Describe some method of meeting this requirement on a long line with heavy traffic.
- [20] 4. A dynamo is fixed on board ship with its axis parallel to the keel. What is the magnitude of the couple which comes on the bearings due to gyrostatic action when the ship pitches? In what plane does this couple act?
- [20] 5. Assuming the usual form of polar curve for the distribution of light from an inverted arc lamp, show how to find the necessary size of reflector to reflect half the total light emitted, assuming the coefficient of reflection to be 80 per cent.
- [20] 6. Describe a Hopkinson test on two direct-current generators. Give diagrams of connections and show how to calculate the efficiencies.
- [20] 7. State briefly the meaning of the following terms:—buffer battery; booster; choking coil; contractor; dead man's handle; root mean square current; surging.
- [20] 8. Describe the construction and arrangement of lightning conductors for a large building.
- [20] 9. Describe the starting arrangements required for a 3-phase induction-motor and a 3-phase synchronous motor. Can they be started under full load?
- [25] 10. Define *power factor*. How does the power factor of the load affect the maximum output of a single-phase generator?
- [25] 11. Describe a suitable design for an overhead transmission line, consisting of three wires 25 square millimetres section each, to carry a 20,000 volt 3-phase current. State the dimensions you propose for the height of the poles, the spacing of the poles, the tension and sag in the wires, the type of insulators; and explain how you would calculate the strength of each pole.
- [25] 12. To what extent can whitewashing the ceiling and walls of a room increase the illumination in it?
Show that the illumination on a table vertically under an incandescent lamp which is hung close to a whitewashed ceiling may be more than double what it would be if the ceiling were black. The walls of the room are supposed to be dark.
How much could the illumination on the table be increased if the ceiling were made of looking-glass?

The Incorporated Association of Municipal and County Engineers.

SUBJECTS OF EXAMINATION.

I.—ENGINEERING AS APPLIED TO MUNICIPAL WORK: 1st Paper:—

- A. Sewage Disposal.
- B. Tramways Construction.
- C. Bridge Construction.
- D. Water Supply.

II.—ENGINEERING AS APPLIED TO MUNICIPAL WORK: 2nd Paper:—

- A. Geodesy.
- B. Hydraulics.
- C. Sewerage.
- D. Road Construction and Maintenance.

III.—BUILDING CONSTRUCTION: STRENGTH OF MATERIALS:—

- A. Materials.
- B. The Construction of Public and Private Buildings.
- C. Building By-laws.
- D. Public Baths and Hospitals.

IV.—SANITARY SCIENCE AS APPLIED TO TOWNS AND BUILDINGS:—

- A. Heating and Ventilation.
- B. Scavenging and Disposal of Refuse.
- C. Water Supply and Drainage of Buildings.
- D. Disinfection.

V.—MUNICIPAL AND LOCAL GOVERNMENT LAW AS RELATING TO THE WORK OF MUNICIPAL ENGINEERS AND SURVEYORS.

NOTE.—The Examiners do not recommend any particular text-books, as it is desired to make the Examinations rather a test of the candidate's practical knowledge of the subjects generally than to find his acquaintance with any particular book or books.

EXAMPLES OF QUESTIONS.

The following questions have been compiled from Examination Papers set to Candidates, and serve as examples of the questions asked under the different sections.

Directions.—"You are particularly requested to write legibly, and to answer the questions as concisely as possible. Fill in your number where indicated, also at the top of every book handed in. Prefix the number of the question to each answer. Place this question-paper inside your book before handing it in. Wherever possible, freehand sketches or diagrams should be drawn to illustrate the answer; these should be carefully executed, as they will be taken as showing the candidate's proficiency in this style of drawing. Candidates must not, during the examination, refer to any books or manuscript, or communicate with each other. Slide rules may not be used."

I. SUBJECT:—ENGINEERING AS APPLIED TO MUNICIPAL WORK.
(Candidates must attempt one question in each section, but not more than six in all.)

First Paper.

(Time allowed 3 hours.)

Section A.—SEWAGE DISPOSAL.

1. Describe briefly the various systems of treatment now in use at outfall works; explain their general principles, advantages, and disadvantages.
2. Describe an up-to-date system of sewage disposal suitable for a district of 10,000 population, taking a dry-weather flow of 40 gallons per head per day, and state how you would deal with storm water. Give dimensions wherever possible.

- [20] 10. Make a sketch of a Joy valve motion, and point out clearly how reversal of the engine is determined by the gear, and how the cut off in the cylinder may be varied.
- [25] 11. It has been found by experiment that, with the regulator wide open and a given cut off in a locomotive cylinder, the mean pressure is a function of the piston speed of the form

$$p = a - cv$$

where a and c are constants, p is the mean pressure in lbs. per sq. in., and v is the piston speed in ft. per minute. Show from this that there is one piston speed at which the horse-power is a maximum for the given cut off, and determine what this piston speed is when $c = 0.068$ and $a = 99$.

- [25] 12. Make a sketch of any one type of axle-box suitable for the main journals of the driving axle of a locomotive. Show clearly the arrangements made for lubrication—or sketch a bearing for one of the crankshaft journals of a high-speed engine arranged for forced lubrication.

Group ii.—METALLURGY.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

- [20] 1. What are the principal refractory substances employed in furnace construction? Give the composition and uses of some of them.
- [20] 2. What methods are adopted for protecting the hearth and bosh regions of blast furnaces driven rapidly with very hot blast? Give sketches of some of them.
- [20] 3. Give some examples of the methods of utilising spent flame heat in reverberatory furnaces both continuous and intermittent in action.
- [20] 4. How is pig iron classified in practice, and what relations are observed between the composition and uses of the different classes? What is the meaning of the term G.M.B.?
- [25] 5. Give a description of the arrangements and method of working of a modern rail mill taking converter ingots without intermediate reheating.
- [20] 6. What are the fundamental principles of "Pyritic smelting," and how has the method been applied in practice?
- [25] 7. A clean iron pyrites (with low siliceous waste) contains 2 per cent. of copper and 2 dwt. of gold per ton. What scheme of treatment should be adopted in order to recover the largest proportion of the constituent values?
- [25] 8. Describe the process of cementation as applied in the production of blister steel and the carburising of armour plates and other finished steel objects.
- [20] 9. Give the composition of some special steels, indicating the uses of each kind.
- [20] 10. What are the properties and uses of nickel, and how is it obtained from pyritic ores like those of Sudbury, Ontario?
- [25] 11. Give the reactions involved in the use of alkaline cyanides in gold extraction, describe how they are applied, and indicate conditions interfering with the realisation of the theoretical consumption of cyanides in practice.
- [20] 12. Argentiferous lead ores containing copper and iron pyrites and zinc blende are specially difficult minerals to smelt. What methods have been proposed for their treatment in practice?

Group iii.—GEOLOGY AND MINERALOGY.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

Illustrate your answers by diagrams.

- [20] 1. Give a general history of mineral veins, explaining their structure, and the mode in which their contents were introduced.
- [20] 2. Define the range of the Boulder Drift in Britain, giving its subdivisions, and explaining its absence from large districts.
- [20] 3. Describe the mode of occurrence in the rocks of gold, iron, silver, radium and

4. A valley has been cut in a plateau along the strike of alternate beds of sand- [20]
stone and shale which dip at an angle of 35° . Show by a diagram how the water
supply is limited in the plateau and in the valley.
5. Describe the circulation of water in the chalk, and explain generally the [25]
conditions of water-supply in a chalk area.
6. Define the various kinds of fault and explain how they have been formed. [20]
7. What is the microscopical structure of clay? Define its chief varieties. [20]
8. Give an account of the geology and physical features of any district which you [25]
have studied in the field, giving lists of fossils and minerals, if any.
9. A district has been selected for a water-supply. It consists of a lake in a rock [25]
basin fed by streams draining an area of 20 square miles. The average rainfall is
60 in. Making reasonable allowance for evaporation, percolation, and a possible
succession of dry years, what is your estimate of the available daily supply?
10. Draw two sections showing the subdivisions of the Jurassic strata in Northern [25]
and Southern England, giving lists of characteristic fossils.
11. Classify the Triassic rocks in Britain. In what subdivisions do you find most [20]
water? Give a list of English towns which are supplied from this source.
12. Describe the conditions of water-supply in Archæan, Cambrian and Silurian [20]
rocks, giving examples in each case.

Group iii.—STABILITY AND RESISTANCE OF SHIPS.

Not more than EIGHT questions to be attempted by any candidate.

The maximum number of marks obtainable for each question is shown in brackets.

1. The lengths of equidistant ordinates of a plane curve are 4.5, 9.1, 14.1, 19.2, [25]
and 23.9 ft. respectively, spaced 20 ft. apart. Find the area and the longitudinal
position of centre of gravity of—
- (a) Whole surface between curve, base line and the end ordinates.
- (b) The portion included between first and second ordinates.
2. A vessel of rectangular cross section is 210 ft. long and 35 ft. broad, and [25]
when floating on an even keel displaces 2520 tons. If then a transverse water-tight
compartment constituting the first 20 ft. of the vessel forward is opened up to the
sea, calculate the final draughts of water forward and aft, the centre of gravity of
vessel being 10 ft. above keel.
3. A vessel of 5000 tons displacement, and having a metacentric height of 2.75 [20]
ft., is observed to keep at a steady angle of heel of 5 degrees under the influence of
wind-pressure on sails. The centre of effort at that angle is at a vertical height of
85 ft., above the centre of lateral resistance; estimate the magnitude of the wind-
pressure in pounds.
4. For what reason is an inclining experiment sometimes carried out on a [20]
vessel?
- A ship had, when inclined, a displacement of 9000 tons and a uniform draught
of 24 ft. A weight of 50 tons shifted transversely 45 ft. caused the bob of a
pendulum 12 ft. long to move 12 in. The transverse metacentre at time of
experiment was 4 ft. above the corresponding water-line; determine the height of
centre of gravity of the vessel relatively to her keel.
5. What calculations are made for a ship of new type to ensure the launch being a [25]
successful one? Show how the results are plotted on a diagram, and explain its use
for the purpose indicated.
6. Why are vessels usually provided with bilge keels? Give in general terms the [20]
results of any experiments with which you may be acquainted, showing the utility of
these keels.
7. Upon what elements of a ship do the times of pitching, rolling and dipping [25]
depend?
8. Define the terms *equivalent girder* and *neutral axis* as used in connection with [25]
strength calculations. Assuming the vessel upright, show how you would arrange in
tabular form the calculation for determining the position of the neutral axis at a
section of the vessel subjected to a hogging moment.

3. A series of settling-tanks are to be constructed, each to contain 250,000 gallons. State the dimensions you suggest for one of such tanks, and give sketch, plan and sections showing the form of bottom you suggest, and the position of inlets and outlets, and how same should be formed.

Section B.—TRAMWAYS CONSTRUCTION.

4. Sketch and describe the various kinds of tramway rail joints in use for electric traction, stating their respective advantages and disadvantages. Design an ordinary fish-plate joint, and point out the features to which you consider special attention should be paid.

5. Sketch the cross section of a road 32 ft. wide between kerbs, one side being 1 ft. lower than the other, with double track tramway, 3 ft. 6 in. gauge; show, in figures, the "canibering" of the road, when paved the whole width with stone setts, or Jarrah wood, and also when the tramway is paved with setts, and the remainder of the road macadam.

6. Sketch a "turn out" or "passing place" on a single line of tramway, figure the leading dimensions, the angle of the crossings, and describe the length, position, and character of the "points" required for diverting the traffic.

Section C.—BRIDGE CONSTRUCTION.

7. Work out the strains on a wrought-iron girder (sketch given) 56 ft. span, 7 ft. high, and give figured sections of flanges, struts and ties. Distributed load 200 tons.

8. State the live load per foot-run of paths and carriage-way you would allow for in the case of a bridge 60-ft. span, with a carriage-way 36 ft. wide, and 12-ft. paths on each side.

The bridge has two lines of tramway, 4 ft. 8½ in. gauge, to carry cars weighing 5 tons when loaded, the distance between the two wheel axles being 18 ft.

9. A bridge has to be constructed to carry a 60-ft. street over a canal, the clear span being 40 ft., the minimum head-room being 11 ft. at centre and 9 ft. at sides above normal water-level: the approaches are rising gradients of 1 in 24 and 1 in 90 respectively. Sketch the bridge you recommend for such a position, giving all important particulars, short specification tests for materials, and tests for bridge when completed.

Section D.—WATER SUPPLY.

10. What percentage of total annual rainfall over a watershed would you expect to have available for storage? State locality and characteristics of the watershed to which your answer relates.

11. A covered reservoir is required to serve a town of 5000 inhabitants. Sketch and describe the reservoir you would adopt, giving all requisite details, and assuming your own conditions of site and foundation. Give a short specification of the necessary works, tests for materials, and an estimate of cost.

12. Sketch and describe a small pumping station capable of lifting daily 100,000 gallons of water from a borehole in the rock and delivering through a rising main 500 yards long into a reservoir, the total vertical lift being 100 ft. After working out the theoretical horse-power required, state the brake horse-power and type of engine you recommend, and give your reasons.

II. SUBJECT:—ENGINEERING AS APPLIED TO MUNICIPAL WORK.

(Candidates must attempt one question in each section, but not more than six in all.)

Second Paper.

(Time allowed 3½ hours.)

Section A.—GEODESY.

1. Describe the mode of making a land survey with the chain only, and with the usual instruments, and explain the advantages of the latter method. Describe

the instruments and give an illustration of a "field-book" with imaginary entries therein.

2. Make a sketch of the primary and vernier scales of a theodolite for reading to minutes. Explain the object of the vernier, and the principle upon which it works.

3. How would you proceed to contour and make a plan of a valley proposed to be used as an impounding reservoir? What is the advantage of a contour plan for this purpose?

Section B.—HYDRAULICS.

4. The velocity in a 9-in. pipe, running full, laid at a gradient of 1 in 48, is 352 ft. per minute. Give the velocities in such a pipe when laid at the following gradients: 1 in 16, 1 in 96, and 1 in 432.

5. A pumping main a mile in length is required to discharge 600 gallons per minute at a velocity of 3 ft. per second. What diameter pipe is necessary?

6. What do you mean by "hydraulic mean depth"? Why is the hydraulic mean depth the same in the case of a circular pipe flowing full or half full, supposing you agree that it is so?

Section C.—SEWERAGE.

7. Describe the usual method of setting out the lines for the construction of a sewer and the means that should be adopted to ensure that the invert shall be laid to the correct depth and gradient.

8. Make a detail sketch with figured dimensions of a storm overflow chamber on a 3 ft. by 2 ft. egg-shaped sewer, discharging when full 600 cubic ft. per minute, assuming that the storm overflow will come into operation when the sewage flowing is one-third the depth of the sewer.

9. A circular sewer, 4 ft. internal diameter, is to be constructed with brickwork in open cutting, the invert being 15 ft. below the surface of the ground. Trial holes show 6 ft. of loose made ground, 4 ft. of clay, and 12 ft. of running sand resting on a thick bed of clay. Show by sketches the timbering of the trench and the construction of the sewer, and give a description of the work and materials.

Section D.—ROAD CONSTRUCTION AND MAINTENANCE.

10. Do you consider the stones in macadam should be all of one gauge, or do you prefer varying sizes? State the reasons for your preference.

11. A macadamised carriage-way 36 ft. wide has got into bad repair, and the whole surface requires to be recoated with stone for an average thickness of 3 in. Explain in detail the different operations necessary to carry out the repairs, including rolling, and give the cost per yard of each operation, assuming that the cost of the stone delivered on the road is 12s. 6d. per ton, binding material 3s. 6d. per ton, and labour 6d. per hour.

12. Give a brief description and express your opinion of any methods of road construction and treatment with which you are familiar for the prevention of dust caused by motor traffic.

III. SUBJECT:—BUILDING CONSTRUCTION.

(Candidates must attempt one question in each section, but not more than six in all.)

(Time allowed 2 hours.)

Section A.—MATERIALS.

1. State what you know of the various timbers used in building, and what class of work and situation they are each adapted for.

2. State the crushing and safe working loads of any brickwork with which you are acquainted, describing the brickwork.

3. Explain, as far as you can, the different characteristics and chemical composition of common lime, hydraulic lime, Roman cement, Portland cement, and Keene's cement.

Section B.—THE CONSTRUCTION OF PUBLIC AND PRIVATE BUILDINGS.

4. A girder, with a clear span of 30 ft., bears a uniformly distributed load of 40 tons; it is supported at one end by a wall and at the other by a hollow cast-iron column, circular in section. The column is 10 ft. in height with fixed ends. Give the dimensions of the column with method of calculation.

5. In the construction of a factory chimney specify the following:—

(a) The subsoil being a stiff clay and weight of shaft 1000 tons, what area of concrete is it necessary to provide for foundation?

(b) What proportion should the spread of the footings bear to the thickness of the brickwork at the base of the shaft?

(c) In a shaft 200 ft. high, give the varying thickness of brickwork from base to top, with distances between the various offsets and height of firebrick lining.

(d) What is the usual proportion of the diameter of base to the height of shaft (circular on plan)?

(e) State the comparative advantages of circular, octagonal, and square shafts with regard to wind resistance.

6. Sketch a concrete beam reinforced with steel rods to carry a distributed dead load of 20 tons with a clear span of 15 ft. Show your calculations.

Section C.—BUILDING BY-LAWS.

7. State how the subsoil of the site of an intended new building should be drained, "where the dampness of the site renders such precaution necessary." Give sketch plan of drains.

8. Describe fully the chief provisions for the prevention of the spread of fire from one house to another.

9. What information must be given to a sanitary authority by a person desirous of laying out a new street?

Section D.—PUBLIC BATHS AND HOSPITALS.

10. Make a cross-sectional sketch of a public swimming-bath 44 ft. in width over all, showing the bath, dressing-boxes, gallery, and roof, with their several dimensions. Describe fully the means adopted for rendering the bath water-tight, and give the type of roof.

11. In designing an isolation hospital for 100 beds, shortly describe the following provisions, viz.—

Number and extent of ward blocks.

Nature of other buildings necessary for administration purposes.

Proportion of floor space per bed.

Proportion of cubical space per bed.

Method of ventilation to be adopted in wards.

Method of heating to be adopted in wards.

12. In designing public baths, comprising, say, a swimming-bath 70 ft. x 30 ft., ten slipper baths, and a small laundry, what method would you adopt to heat the water, and how would you warm the building?

IV. SUBJECT:—SANITARY SCIENCE AS APPLIED TO TOWNS AND BUILDINGS.

(Candidates must attempt one question in each section, but not more than six in all.)

(Time allowed 3 hours.)

Section A.—HEATING AND VENTILATION.

1. Explain what you mean by "natural" and "artificial" ventilation. Illustrate your answers by sketches of each as applied to a public building or school. What are the advantages and disadvantages of each system?

2. In preparing a scheme for warming a building, what are the recognised methods of calculating the amount of heating surface required for varying temperatures?

3. Describe concisely three different methods of ventilating sewers, expressing your views as to their merits or otherwise.

Section B.—SCAVENGING AND DISPOSAL OF REFUSE.

4. Describe the method you would adopt for collecting—

(a) The contents of cesspools,

(b) Excreta in pails,

(c) Offal.

and give particulars of the vehicle you would recommend in each case.

5. Describe the essential features of a good type of refuse destructor, and give a section of the same.

6. Describe the most satisfactory means of

(a) Removing house refuse,

(b) Cleansing a-linits and privies,

where the local authority undertake the work of removal.

Section C.—WATER SUPPLY AND DRAINAGE OF BUILDINGS.

7. Describe the construction of a hot-water service for bath and other purposes in a house. Describe how the circulation is obtained, and show by a diagram the position of the boiler, cistern, cylinder, etc., and state the precautions that should be taken to prevent damage by frost.

8. State shortly what are the general principles of efficient house drainage, and give a few examples of how these are frequently disregarded in actual practice.

9. What kind of water acts upon lead? State the risk attendant upon its use, and the means you would adopt for avoiding or reducing such risk.

Section D.—DISINFECTION.

10. Mention the various disinfectants in common use, and state which you consider the best for particular purposes. Describe the methods of use and the precautions necessary.

11. What are the points to be observed in the selection of the necessary apparatus to effect the proper disinfection of bedding and clothing? Give a brief description of the apparatus with which you are acquainted.

12. Describe how you would proceed to disinfect a house after the following diseases: (a) scarlet fever, (b) typhoid fever, (c) diphtheria, (d) small-pox.

V. SUBJECT:—MUNICIPAL AND LOCAL GOVERNMENT LAW AS RELATING TO THE WORK OF MUNICIPAL AND COUNTY ENGINEERS AND SURVEYORS.

(Candidates must not attempt more than six questions.)

(Time allowed 2 hours.)

(ENGLAND.)

1. Under what Act can a local authority make by-laws as to deposit of plans for alterations to existing buildings, and to what buildings do these by-laws apply?

2. What are the conditions under which a manufacturer may discharge his trade refuse into the sewers of a local authority?

3. Can a local authority compel a proper supply of water to be laid on to premises, and if so, under what conditions in (a) urban districts, (b) rural districts? State the mode of procedure in each case.

4. The surveyor to a local authority has reported to him (a) a dangerous building abutting on a public highway, and (b) a dangerous chimney on private enclosed

premises. State what are his powers and duties in each case, and set out fully the proceedings he would take in proper sequence.

5. An occupier refuses to allow his premises to be entered by the assistants of the surveyor to a local authority for the purpose of taking levels: what course is prescribed by statute to meet such a case?

6. State which Acts give powers respecting tramways and light railways, and describe the principal differences between them, particularly as to position of tramway in road, spaces, notices, supervision, and mode of procedure.

7. Describe the provisions of the Buildings in Streets Acts, 1888, and point out in what respect they differ from the previously existing powers.

8. What rights have owners and occupiers of property to connect their drainage with the sewers of a local authority (a) within the district, (b) without the district, (c) under what restrictions can the connections be made, and (d) what are the penalties for non-compliance?

9. Give two instances where you consider the present Public Health and Sanitary Acts or Highways Acts are defective or require amendment, and in what way would you remedy these defects?

This question is intended to elicit answers from candidates as to any difficulties they may have experienced or observed or heard of in carrying out the duties of a surveyor.

10. What is a Provisional Order, and when is it usually applied for? Compare it with a local Act of Parliament, specifying its chief advantages. State the nature of Provisional Orders issued by the Board of Trade as distinct from the Local Government Board.

11. Give reasons for recommending a council to adopt the Private Streets Works Act, 1892. State also the disadvantages of that Act compared with sections 150 and 152 of the Public Health Act, 1875.

12. What powers have local authorities with respect to—

- Planting of trees in highways?
- Underground conveniences?
- Sanitary conveniences for manufactories?
- Ingress to and egress from places of public resort?
- Safety of platforms on public occasions?

V. MUNICIPAL AND LOCAL GOVERNMENT LAW AS RELATING TO THE WORK OF MUNICIPAL ENGINEERS AND SURVEYORS.

(Candidates must not attempt more than six questions.)

(Time allowed 2 hours.)

(SCOTLAND.)

1. Define the various roads to which these words apply, viz., Highway, Turnpike Road, Statute-Labour Road, as interpreted by the Roads and Bridges (Scotland) Act, 1878.

2. Describe the statutory provisions whereby a local authority can recover from any person expenses for damage to highways caused by extraordinary traffic thereon, or by excessive weight passing along the same; and state in what Act of Parliament these provisions are embraced.

3. To whom must application be made for authority to lay out new streets? Specify the details which require to be shown on the plan accompanying the application.

4. If a Burgh should desire to improve any existing private streets, what statutory procedure would be necessary?

5. What are the powers of a Burgh with reference to the keeping of footpaths of public streets in proper repair? How far do these powers apply to private streets?

6. Specify the procedure which must be adopted in a Police Burgh before a public sewer can be laid. Under what Act is this necessary?

1. What powers of entry are given under the Public Health (Scotland) Act for the purpose of examining drains, and what is the necessary procedure?

2. Specify the duty of a Local Authority with reference to the water supply of buildings in an isolated district.

3. Enumerate the powers given for the formation of special water-supply districts, and specify under what circumstances a Local Authority is bound to take action.

4. What were the requirements of the 1892 Burgh Police (Scotland) Act with regard to back space for proposed buildings, and what alteration was made by the 1904 Act?

THE ROYAL SANITARY INSTITUTE.

In the Royal Sanitary Institute Examinations there is only really one division which has an important aspect to the civil engineer—that bearing upon Sanitary Science as applied to buildings and public works.

SYLLABUS OF SUBJECTS.

It is essential that a candidate should have a practical as well as a theoretical knowledge of the subjects set out in the Syllabus.

ELEMENTARY PHYSICS AND CHEMISTRY.

in so far as they apply to Sanitary Science.

The principles of hydraulics and hydrostatics, pneumatics and heat, and the composition and properties of air and water.

The diffusion and movement of gases and liquids.

The velocity and discharge of liquids and gases through pipes and orifices.

Porosity, capillarity, absorptivity, permeability.

Definition of vacuum, action of siphon, principle of suction, force, and lift pumps.

Production and transmission of heat, and effects of heat on solids, liquids, and gases.

The pollution of air and its effects, and the principles involved in warming and changing air.

The solvent effect and action of substances in solution, the principles involved in softening, filtering, and purifying water.

The principles involved in the various methods of treating sewage.

LOCAL CONDITIONS.

Aspect—elevation.

Hill, plain, and valley.

Distance from sea.

Influence of surrounding objects.

Winds—rainfall—humidity.

Soil and subsoil, and its drainage.

Pollutions of soil.

Sanitary precautions as to healthiness of site.

Ground air and ground water and their pollution.

WATER.

Source of supply.

Physical characteristics of various waters.

Requirements and supply of towns, villages, country houses, and colonies.

Methods of storage and storage.

Methods of contamination and protective precautions.

Distillation, softening, and other purifying processes.

MATERIALS AND CONSTRUCTION.

General description of materials used in construction, namely, masonry, timber, concrete, stones, bricks and tiles, terra cotta, earthenware, cement, Portland materials.

for covering roofs, and materials used for preservation and decoration, such as rendering, plaster work, paints, and varnishes; as to their—

Perviousness to moisture, conductivity of heat, facility of working and using, strength, durability, power to withstand fire, purposes for which suitable, relative cost.

General principles of construction as applying to—

Foundations in various soils.

Walls; hollow walls, damp course, bond.

Floors for basements or cellars, fire-proof partitions.

Roofs, covering for same, gutters, hips, valleys.

Fire-proof construction.

Principles of calculating areas, cubic space.

Interpretation and drawing of plans and sections to scale.

AIR, LIGHTING AND WARMING.

Air movements.

Sources of pollution.

Principles of ventilation.

Air space and quantity required.

Methods of, and appliances for, ventilation.

"Overcrowding on space" and in buildings.

Air space surrounding buildings.

Angle of daylight illumination.

Methods of artificial lighting, advantages and disadvantages.

Size of windows.

Open fires, stoves, hot water, steam, hot air.

Ventilation of public buildings, hospitals, schools, factories, and dwelling-houses.

DRAINAGE, SEWERAGE AND SANITARY APPLIANCES.

The proper conditions of good drainage.

Re-modelling old drainage.

The planning and construction of new drains and sewers.

Disposal of surface and rain water.

Advantages and disadvantages of various sanitary appliances.

Drain testing and inspection of drainage work and appliances.

The various systems of dealing with house sewage and house refuse.

Disposal of sewage and refuse.

Disinfecting apparatus and disinfectants.

EXAMINATION QUESTIONS.

June, 1905.

1. What do you understand by the relative humidity of the atmosphere, and how is it measured? Describe in detail any hygrometer with which you are acquainted, and the method of using it.

2. Show by sketches sections through the basement walls and floor of a house constructed on a wet site; the basement is 9 ft. high, the floor level is 6 ft. below the surface of the surrounding ground. What material would you use for the damp-proofing, and how would you construct the basement floor?

3. What do you understand by (1) a shallow well, (2) a deep well, (3) an artesian well? What is the fundamental characteristic of each, and how does the water from them commonly differ?

4. How are doors and windows made fire-resisting? and what other precautions can be taken to prevent the spread of fire in large buildings?

5. Explain what is meant by "overcrowding on space" and in buildings. Write a short paper on the difficulties local authorities have in connection with the housing question.

6. What do you understand by the terms "radiation," "conduction," and "con-

vection"? Give examples as applied to heating rooms. Enumerate the various methods of warming rooms with which you are acquainted, and describe their advantages and disadvantages.

7. Show by sketches the arrangements you would suggest for purifying the sewage from a house situated in an isolated position on a clay subsoil, and on a gravel subsoil; there is a stream close by into which the effluent may be discharged.

8. What method of excrement and refuse disposal would you recommend for a roadside cottage where there is no public water supply or public scavenging? What precautions would be necessary if the only available land was at the rear, and on a higher level?

APPENDIX XI.

SCAFFOLDING.

THERE are two varieties of scaffolding in common use.

1. The bricklayer's scaffold.

2. The mason's scaffold (or independent). There being some difference between the two, considering the first-named type:—

The scaffold poles are first erected and planted in the ground or other firm foundation, a common practice being tubs filled with earth. They are usually placed 8 ft. centre to centre, and 4 ft. from the wall to be built. They are termed the standards. Then at about 5 ft. 6 in. to 6 ft. from the ground-level are placed horizontal poles termed ledgers, which are lashed to the scaffold poles (standards) or fixed by special clamps. As the wall proceeds, bricks are left out in places and pieces of timber 3 in. x 3 in. x 5 ft. 6 in. are inserted, being set level and resting on the ledgers at the ends. They are called the "put logs." Upon the put logs are placed 9 in. x 1½ in. x 12 ft. spruce boards for the men to work on, the ends being usually iron-bound to protect them from damage. Double boarding is used above public footways to prevent dust falling. Gantries are preferable for this work (figs. 1020 and 1021). In the mason's scaffold we have closer standards and a very much stronger scaffold, and there are also two frames, one 4 in. from the wall and the other 4 ft. as before. The frames are also cross-braced and the put logs lashed to the ledgers.

APPENDIX XII.

CONSTRUCTING A RAKING SHORE.

For methods of design *vide* Chapter XVII.

The first thing to do is to lay a 9 in. x 3 in. wall plate against the wall. This is to distribute the pressure over an uniform area. This is fixed to resist sliding by a small piece called a "needle,"* which is usually a piece of wood 1 ft. x 4 in. x 3 in. passed through a mortise cut in the wall-plate, half a brick having previously been removed from the wall for the purpose. The needle should project about 4 in. The plate is then fixed by hooks driven into the brickwork. The sole-plate is next considered. It is laid on the ground, having a firm bearing, and being carefully set at the correct angle, which is usually about 87° to the centre line of the shore. Having then cut the shores to the proper angle by means of the bevel-square, they are placed in position. A common practice is to provide wedges, and to raise or lower the shore by hammering these home or releasing them. This is a practice, however, which, by reason of the vibration set up, is best avoided, and to do so a small groove is cut in the base of the strut to allow of this being done by crowbars.

Finally, the struts are put in position and the needles finally set immediately below the wall plates.

* Sometimes called a "Joggle" or "Tossle."

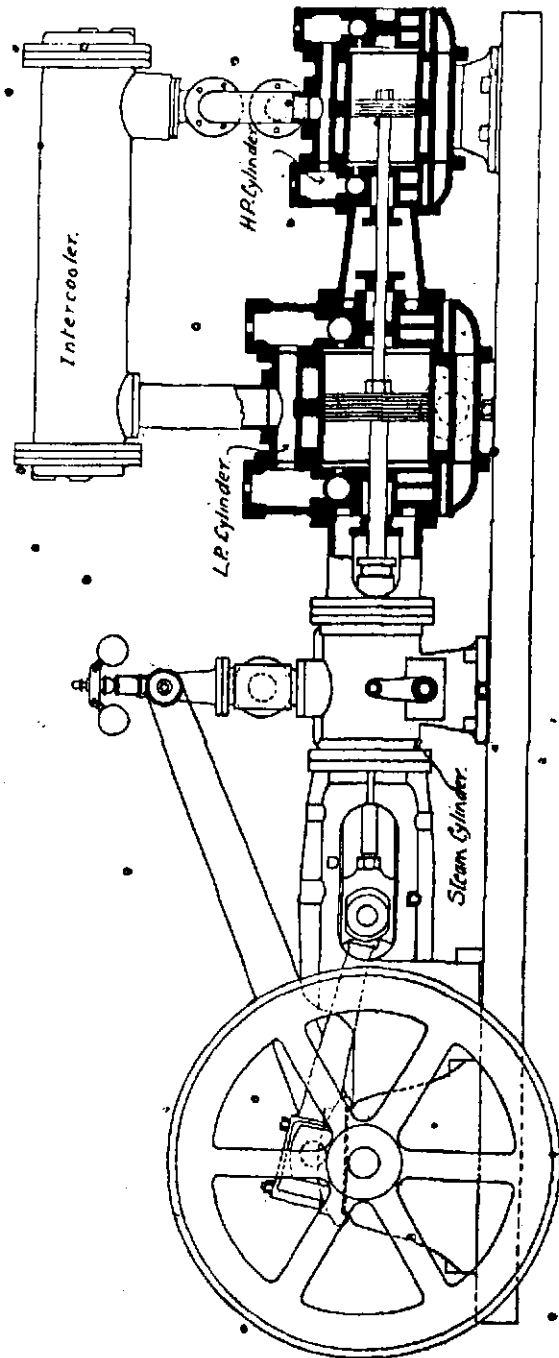
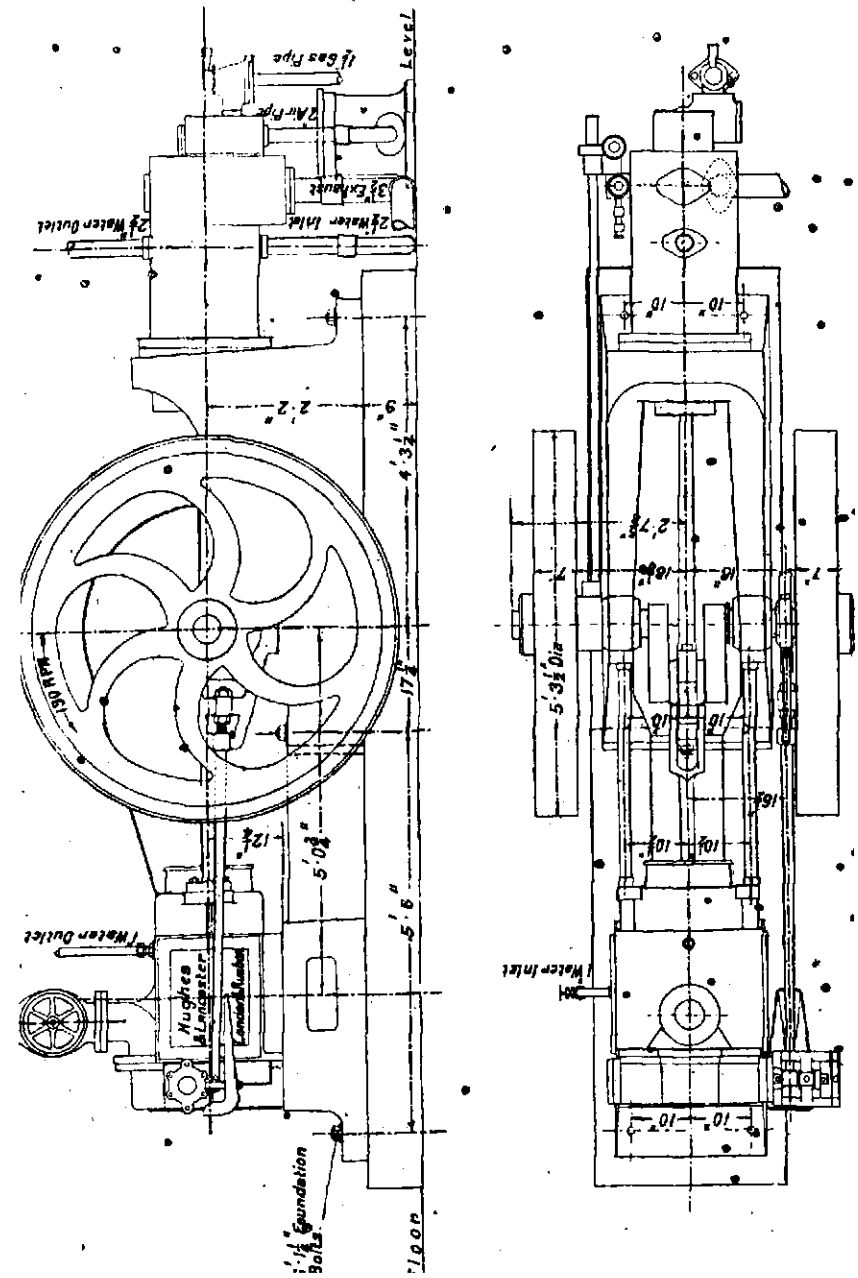


FIG. 1066.



APPENDIX XIII.

COMPRESSED AIR.

We have already touched upon the use of compressed air in an earlier part of this book in connection with the raising of sewage by means of Shone's ejectors. The use of this means of power transmission in sewerage works continues to find much favour amongst engineers, a fact which has led the writer to add a few words on the actual practice of air compressors. The first important consideration is the elevation above sea-level which the compressor is required to work at, because, due to the consequent variation of atmospheric pressure, the output of any compressor of given dimensions will vary, those at the lower elevations giving the greatest efficiency.* Also at a constant temperature the volume will decrease directly as the pressure (Boyle's law). Hence the first aim of compressor designers is to keep the percentage of clearance between the piston and cylinder covers at the dead centres as small as possible, because all the air entrapped in this space has to expand to initial atmospheric pressure (at least) before any admission can take place, which will, of course, affect the volumetric efficiency, because admission cannot commence before the piston has travelled some distance (about 10 per cent.) of its stroke. Many makers of compressors make no allowance for this, a fact which wants very careful investigation by the engineer. Hence we see that the long-stroke small-bore compressor is the best for a given volume. For small installations, single-stage machines are invariably used; probably only very large schemes will warrant the expense of a two-stage compressor. The limit of the single-stage machine would be about 200 cub. ft. of air per minute at a receiver pressure of 100 lbs. per sq. in. When any single unit is required to give greater duties than this, it is generally considered prudent and economical to install a two-stage machine having an intercooler. Another point which it is very important to bring before those whose experience with compressed air is only small is, that the piston rings require very careful fitting, more so than the steam cylinders. It is a common thing to see a new machine giving almost inconceivably good indicator cards from the air cylinder, that is, very small diagrams, with which the inexperienced engineer would be agreeably satisfied, when perhaps all the time a leakage was taking place past the piston rings with this result. The reason for the desirability of small diagrams is because they show that the compression is approaching isothermal (viz. having an equal degree of heat), and this is what all designers are aiming at. The smallest amount of work required to compress 1 lb. of air to any pressure will be smallest when the compression approaches isothermal. If none of the heat is absorbed, however, the curve rapidly approaches the adiabatic.

The equation for this curve is

$$PV^n = \text{constant} \quad (517)$$

$n = 1.406$ for adiabatic and 1.0 for isothermal. Then, besides showing a small diagram, leakage past the rings will also cause a very uneven curve.

Referring to the diagram fig. 1064, we have the total work done in ft.-lbs. in compressing 1 lb. of air.

(1) Adiabatic compression

$$\begin{aligned} &= \text{if } PV^n = \text{work during delivery} \\ &\text{and if } P_0 V_0 = \text{work during suction} \\ &\text{and } n = 1.406, \end{aligned}$$

$$\text{then } W = \frac{PV_0 - P_0 V_0}{n-1} + PV_0 - P_0 V_0 \quad (518)$$

But if the work is isothermal,

$$W = C_1 \log \frac{P}{P_0} + PV_0 - P_0 V_0 \quad (519)$$

PV_0 referring to fig. 1065 and = work during delivery.

* Note.—The reduction of capacity is about 3 per cent. for every 1000 ft. above sea-level.

For large volumes of air at fairly high pressures much can be said for the two-stage machine. When, however, the pressures are comparatively low, such as for blast furnaces, etc., many engineers are now installing turbo-blowers, working on the principle of a reversed turbine. Where we have long lines of transmission, however, it is essential to keep the velocity of air below 40 ft. per second; the friction between the air and the pipe also varies inversely as the diameter; hence pipe lines must be designed within reasonable limits on these considerations. On small machines automatic poppet valves are nearly always used. They should be many in number and small in diameter. They should only have a small lift, those on the delivery side having the same total sectional area as the suction side. This, however, is not always done in practice. No stout or stiff springs should be allowed on any valves; they should be as long and light as possible, and preferably double, and be easy of access. Those on the delivery side must be capable of remaining open till the end of the stroke. They should then close rapidly, and open again without noise. In large machines, however, like those of the well-known Worthington type, positively operated Corliss valves are now used, because it is essential for efficiency that the air entrapped re-expands to atmospheric pressure before the inlet valve is opened. This is not always obtainable with poppet valves, but Corliss valves admit of the adjustment to a nicety. A two-stage compressor with Corliss valves in the cylinder covers is shown

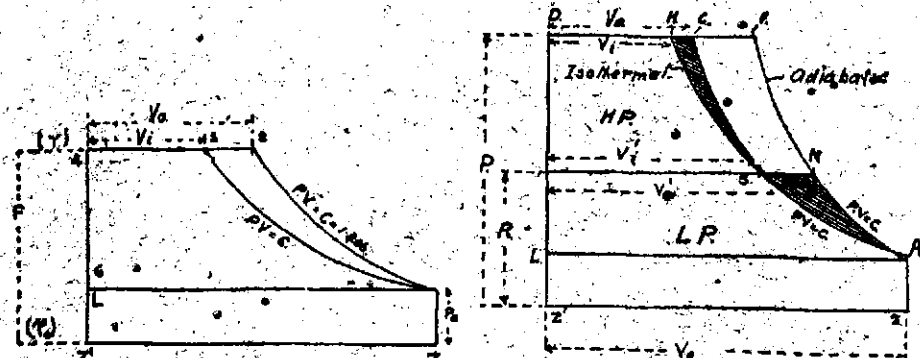


FIG. 1064.

FIG. 1065.

in fig. 1066, while figs. 1067 and 1068 show an ordinary gas-driven compressor specially adapted to use with Shone's ejectors for drainage work. When a two-stage compressor is used, the diagram in fig. 1064 becomes altered to fig. 1065, the space SN showing a reduction in volume from V_0 to V_1 due to the intercooler NFCS, being the area of work saved by the intercooler.

We proceed to investigate how mechanical energy is absorbed in compressing air. For instance, say we are going to consider compressing 1000 cub. ft. of free air to 100 lbs. pressure per minute, using a two-stage machine. The first thing to do is to find the mean effective pressure in the air cylinders

$$= \text{M.E.P.} = \frac{n}{n-1} \times x \times P_0 \left\{ \left(\frac{P_c}{P_0} \right)^{\frac{n-1}{n}} - 1 \right\} \quad (520)$$

n = a coefficient = 1.25.

x = number of stages = 2.

P_0 = absolute pressure at inlet valves.

P_c = absolute pressure at delivery valves } lbs. per sq. in.

$$\therefore \text{M.E.P.} = \frac{1.25}{1.25-1} \times 2 \times 14.7 \left\{ \left(\frac{114.7}{14.7} \right)^{\frac{1.25-1}{1.25}} - 1 \right\}$$

$$= 5 \times 2 \times 14.7 (1.2281-1)$$

$$= 33.53 \text{ lbs. per sq. in.} = \text{M.E.P.}$$

and is set off to scale as in fig. 1072. We assume, however, in this case that we have no snatch block. Now AB = vertically the load to scale and BC = pull on chain (viz. from c-d). Joining AC, we have the direction and magnitudes of the resultant to the two chain pulls at the jib-head. We then complete our triangle by drawing AC and DC parallel to the tie-rod and jib respectively. Then by scaling off we have on AD 23 tons and DC 40 tons, the accuracy of these values depending on the scale to which the diagram has been plotted. For a crane of this description we would want a 28-ft. jib, and when considering design we have a pin-ended strut (vide Chapters IX. and X.). A large factor of safety must be allowed, as the stresses will naturally be very sudden. Again, if L = length and r = radius of gyration, we must find a suitable value for $\frac{L}{r}$. Make it, say, 100, and let us try a pair of 9-in. $\times$ 3 $\frac{1}{2}$ -in. $\times$ 25-lbs. channels. We proceed to use the Rankine-Gordon formula

$$P = \frac{50000}{1 + \frac{L^2}{18000 r^2}}$$

18,000 being based on the fact of having two pin-ends.

$$\therefore P = \frac{50000}{1 + \frac{(28 \times 2)^2}{18000 \times 3.43^2}} = 49,000 \text{ lbs., say,}$$

as the ultimate strength in lbs. per sq. in.

Now the area of our two channels = 7.47×2 = say 16 sq. in., and $49,000 \times 16$ = 784,000 lbs., and $\frac{784000}{40 \times 2240}$ = a factor of safety of 8.75, which would do well. The channels would have their flanges laced together by 3-in. $\times$ $\frac{1}{2}$ -in. flats, both top and bottom. Batten plates would be provided at top and bottom of the jib. Then we find that our selection will satisfy our previous requirements of $\frac{L}{r}$ = 100, because

$\frac{28 \times 12}{3.43} = 98$, and the maximum distance between the lattices, which = 98×1.01 (1.01 = minimum radius of gyration of a 9-in. $\times$ 3 $\frac{1}{2}$ -in. channel) = 8 ft. 3 in., would be easily diminished in practice. Calculating, we find the jib = about 1 ton in weight. We proceed to resolve the weight into a direction at right angles to the jib as in fig. 1072, which gives us about 6 ton in compression and 7 ton overturning. The B.M. = $\frac{WL}{8}$

= $\frac{6 \times 28 \times 12}{8}$ = 25 tons-inches. Now $\frac{\text{B.M.}}{\text{mod.}} = \frac{25}{39}$ = 6 tons, easily covered by the factor of safety. Channels, it may be remarked, would make a very neat jib. They are, however, especially in shear legs and other heavy machinery, made of 4 angles, laced and swelled out at the centre. Let us try four 3 $\frac{1}{2}$ -in. $\times$ 3 $\frac{1}{2}$ -in. $\times$ $\frac{1}{2}$ -in. angles, swelled to 16 in. square at the centre. The approximate radius of gyration = $\frac{16 - 2}{2} = 7$ in., assuming that the centre of gravity of the angle is 1 in. from the back. In this case, however, we only have the ratio $\frac{L}{r} = \frac{28 \times 12}{7} = 48$. Using the previous formula for this section we have $P = 44,300$ lbs., while the area of the 4 angles = $4 \times 3.25 = 13$

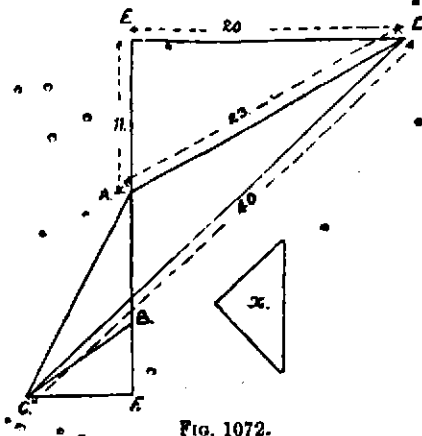


FIG. 1072.

$\therefore$ safe load = $\frac{44300 \times 13}{6} = 42$ tons, assuming a factor of safety of 6. In fig. 1072, if we draw ED and EA horizontal and vertical as shown, we have to scale ED = 20 tons and EA = 11 tons. Again, FC drawn horizontally = horizontal pull on the chain, while FB = its vertical component. Also if we want to find the diameter of the post from our results we proceed as follows: Let D = diameter of post, W = pull on top of post, and S as shown in fig. 1071, and K = a coefficient = 22.

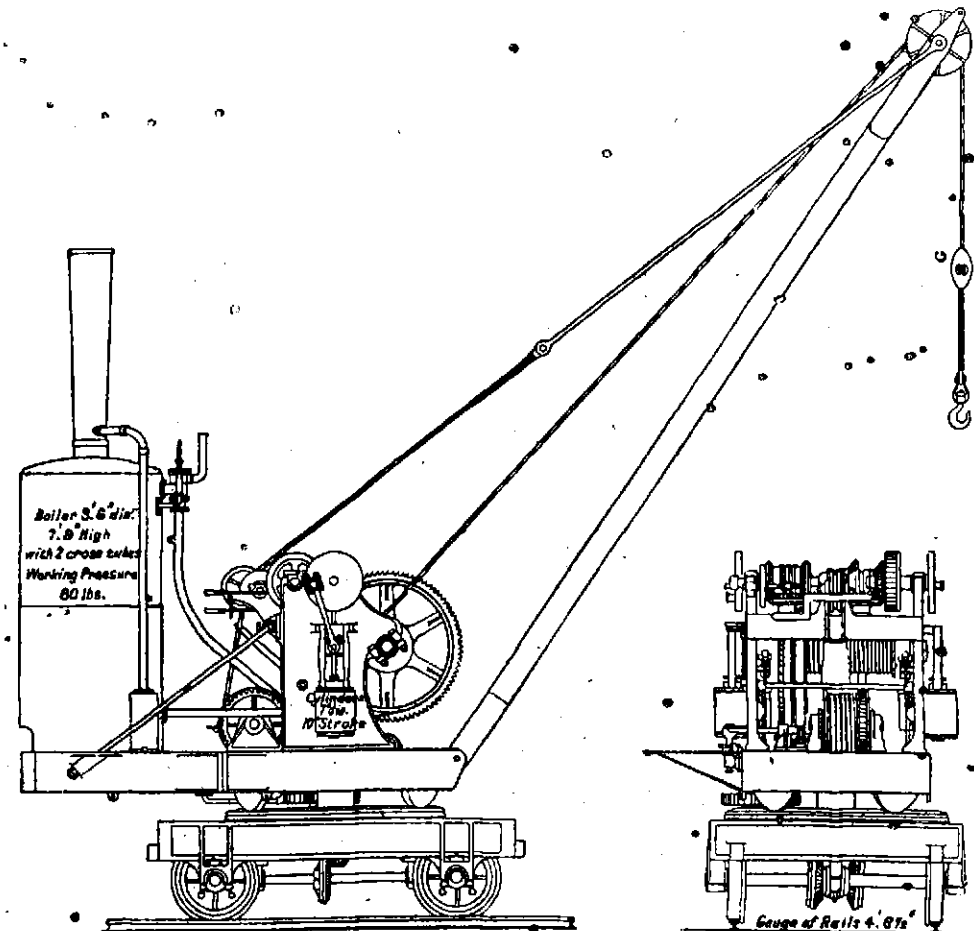


FIG. 1073.—Three-ton Locomotive Steam Crane.

Then

$$D = \sqrt[3]{KWS} \quad (521)$$

$W = FC = 8$ tons at a height of 3 feet. Resolving, we have an equivalent pull at the top = $\frac{8 \times 3}{7} = 3.4$ ton = a total pull of $3.4 + DE = 3.4 + 20 = 23.4$. $\therefore D = \sqrt[3]{22 \times 24 \times 7} = \sqrt[3]{3696} = 15\frac{1}{2}$ in. diameter. Now the upward pull $EA + BF = 11 + 5\frac{1}{2} = 16\frac{1}{2}$ tons. This is partly resisted by the thrust of the jib + weight of crane gearing. Assuming

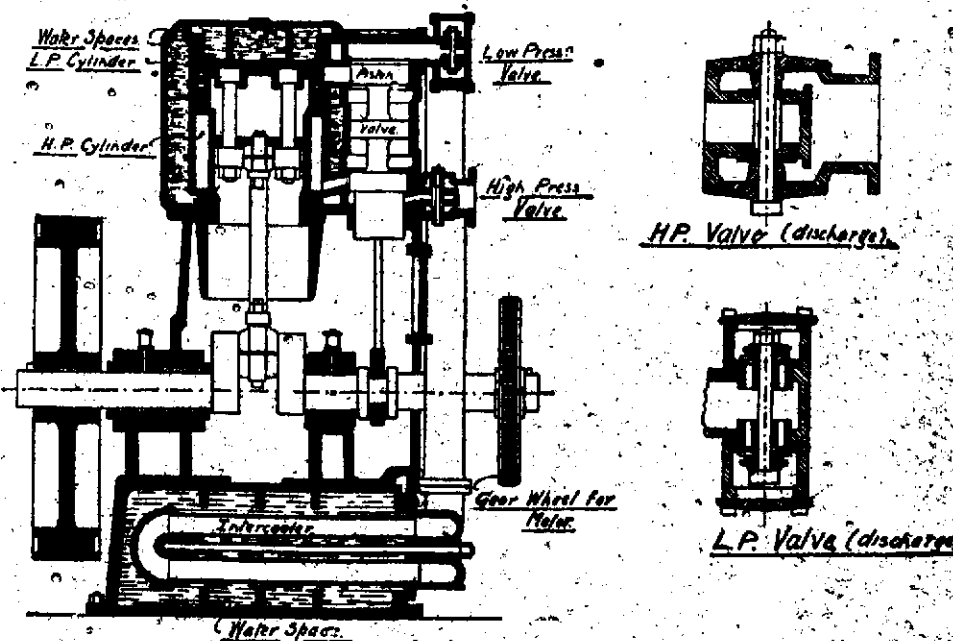
The I.H.P. is then easily calculated by the well-known equation

$$\text{I.H.P.} = \frac{\text{PLAN}}{33000} \times 2,$$

except that we are considering cubic feet of air, whereas A is taken in sq. in. Hence the volume

$$= 2 \times \text{stroke (feet)} \times \frac{\text{Area of piston (sq. in.)}}{144} \times \text{R.P.M.}$$

But this is equal to the displacement; hence if it is considered as cub. ft. per minute,



Figs. 1069 and 1070.

$$\text{I.H.P.} = \frac{\text{displacement} \times 144}{33000} \times \text{M.E.P.}$$

and 144 divides into 33,000 229.1666 times.

$$\therefore \text{I.H.P.} = \frac{\text{displacement} \times \text{M.E.P.}}{229.166}$$

We now want to find what the volumetric efficiency of the compressor is likely to be. We can take it safely at 90 per cent. Hence the displacement will be equal to the capacity $\div$ 90 per cent. $= \frac{\text{capacity}}{0.90}$, the capacity being of course in cub. ft. per minute.

$$\therefore \text{I.H.P.} = \frac{\text{capacity} \times \text{M.E.P.}}{0.90 \times 229.166}$$

But we know M.E.P. = 33.63 $\therefore$ as $33.63 \div 229.166 = 1625$, I.H.P. = capacity $\times$ 1625.
The I.H.P. required = $1000 \times 1625 = 1625$. Assuming it to be steam-driven, Cu

uld take an efficiency of 85 per cent., giving a maximum horse-power required of out 200. For a gas engine this would be about 210 if the horse-power was reckoned on the indicator diagram, or about 170 if on the brake.

A very neat design of electrically driven compressor, adaptable to sewage pumping, shown in figs. 1069 and 1070. Such machines as these would work without mediate attention, beyond inspection, cleansing, and oiling, by means of an automatic switch controlled by the receiver pressure, which would ensure marked economy, especially if electrical power could be cheaply obtained, which is a very likely occurrence, considering both the sewerage and electrical undertakings would probably belong to the same public body.

APPENDIX XIV.

DESIGNING JIB CRANE MEMBERS.

As we have previously made mention in Chapter XII. of the methods whereby the stresses set up in the members of a crane under load can be ascertained, although

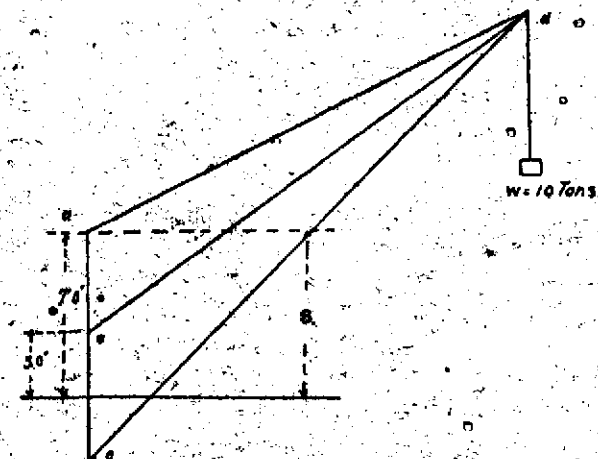


Fig. 1071.

the design would be very foreign to this book; yet the civil engineer might at any time, especially in work abroad, be called upon to design the jib and other members of a particular type of engine which happened to be on the works, and to adapt to certain circumstances, or to alter an existing crane likewise, so as to avoid the expense of a new machine and loss of time in waiting for it. The writer has more than once had to turn his ingenuity to account in such a case. We will therefore give a practical example. It is supposed that an engine and boiler, crane-post, etc., is ready, and it is proposed to turn it to account by making a jib 20 ft. high from jib end to sheave, lifting 10 tons at a 25-foot radius, the gearing, roller-paths, and mechanical details not being considered; as they already exist in working order. Considering our case graphically, in fig. 1071 we have a parallel to this tie-rod, a parallel to the jib, and a parallel to the chain or rope which leads to the barrel, the point a being at the height on the centre line of the post where the line of the lifting chain cuts it if produced. The base line is drawn to represent the centre of the crane, which supports the post at its mid-diameter. If the crane has a snatch hook for the full load, the stress on the chain will be reduced by one half = 5 tons,

that it all comes on the post, we have an additional area of $\frac{18.5}{4} = 4.1$ sq. in., which would increase the post diameter by $\frac{1}{4}$ in.

The principal types of crane which the civil engineer deals with are usually the derrick crane and the small vertical locomotive crane of anything up to 10 tons capacity. The former are made in all sorts of different designs. A gantry (or staging) for such a crane was previously illustrated in Chapter XVII. The jibs are nearly always made on the principles previously dwelt upon, and are often of huge dimensions as regards the length, although their actual construction would be very light. They are extremely useful for the erection of heavy masonry and steel buildings on land. For harbour works, drainage, waterworks, and water mains, etc., also canals and railways, the locomotive crane with the vertical boiler has found great favour. A typical example is shown in fig. 1073, which was supplied to me by Messrs J. H. Wilson & Sons of Birkenhead. The jibs of these cranes are usually made of timber, sometimes strengthened by iron plates. The type of crane shown is very adaptable to circumstances, and is a useful machine. The vertical boiler is, however, wasteful of coal, and will not keep the steam pressure constant, facts, however, which are of almost minor importance in such cases. They may frequently be bought second-hand, often in good condition too, and quite good enough for rough usage on contracts, provided they are in good working order; the only thing which is absolutely necessary is to have the boiler rigidly inspected. This is usually done by experts; in cases, however, if the work is to be done by the resident engineer or other responsible official, the following hints will be of use:—



FIG. 1074.

FIG. 1075.

FIG. 1076.

Defects in Crane Boilers.

To examine the interior of the boiler, first remove the doors from the manhole, mudhole, and handholes, and see that the interior is thoroughly dried, which may be done by lighting a small wood fire in the furnace. Unless the boiler is an uncommonly small one it is not a difficult matter to get inside, though the attitudes to be assumed when there may be somewhat cramped. It will be necessary to have a light, and a candle will be found the most convenient, as it can be thrust into narrow spaces where a lamp would be useless. A light hand-hammer should also be taken, preferably one with a wedge-shaped head or with a chisel point at one end. Examine first the shell plates, especially along the water-line; in many cases there will be signs of corrosion at this part, more or less pronounced, while the remainder of the plate, above and below, is in good condition. If the corrosion does not extend to a depth of more than $\frac{1}{16}$ in. it will not materially affect the strength of the shell, owing to the large factor of safety usually adopted for boiler construction. If the boiler is similar to one of those shown in fig. 1073, the uptake in one case and the tubes in the other must also be carefully examined at the water-line. Corrosion in boilers usually takes the form of "pitting"—that is, circular or oval patches measuring from $\frac{1}{4}$ in. to 2 in. across, which can easily be detected at sight or by passing the hand over the affected part. There is, however, one very deceptive form of corrosion in which the entire exposed surface of the plate is evenly affected, with the result that to all appearances the plate is in good condition. In this case the only way in which the true state of affairs can be ascertained is by examining the seams at the edge of

the overlapping plates. The appearance of a plate attacked by general corrosion is shown in fig. 1074, in which the dotted line indicates the original surface of the plate. When it is not possible to see the seam, the defect may be discovered by running the finger-tips along the edge of the plate.

The tubes of all boilers fitted with such will require careful inspection for signs of pitting. These tubes are only about $\frac{1}{2}$ in. thick, so that in this case corrosion to the extent of $\frac{1}{16}$ in. would render them positively dangerous. If the tubes are covered with scale or incrustation a portion of it should be removed from two or three tubes to expose the surface of the metal beneath and note its condition. It is not advisable to remove the scale from the whole of the tubes, as is sometimes done, because, provided the scale does not exceed $\frac{1}{8}$ in. in thickness, it forms an efficient protection against corrosion, and if any pitting has taken place before the scale was formed, the marks will be plainly visible, owing to the deposit of scale being of even thickness throughout.

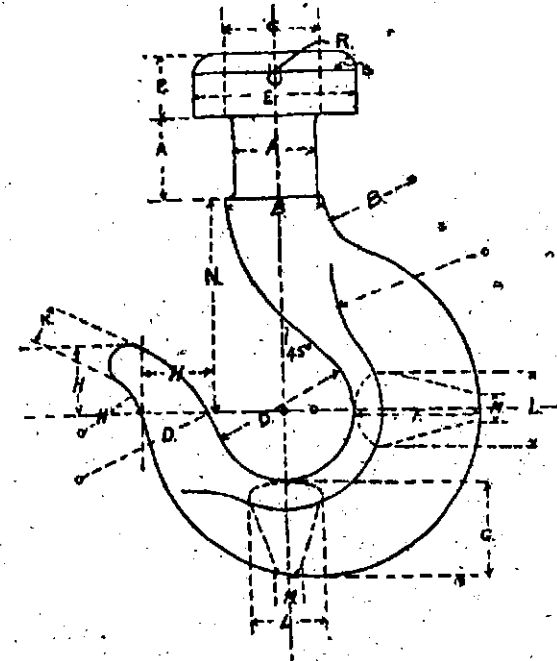


FIG. 1077.

All fittings should be examined. It may be here remarked that all machinery used on civil engineering contracts should have a great deal more care bestowed upon its proper maintenance and upkeep than is generally done. The engineer should make it his duty to see that neglect, and consequent liability to breakdown, is not permitted. These remarks refer, of course, to work done by direct labour.

In concluding our remarks on cranes, it may be well to show how the hooks are usually designed, because in out of the way places a failure may necessitate the engineer having one made at the local smithy.

The common type of crane hook up to 30 tons capacity is shown in fig. 1077. The form of cross section at the various points is shown in dotted lines, while the standard dimensions adopted by engineers are shown in the table in reference to the shattering, according to the maximum load it is proposed to lift. For larger cranes, not often found on civil engineering works, a form known as the ram's head shackle is used, or the closed triangular type.

[illegible]

LEAP WEIRS.



Plan of C. Iron Key

Masonry

Concrete

This diagram shows a cross-section of a circular structure, likely a well or a large pipe, with a central circular opening. The structure is composed of several layers: an innermost layer of masonry (hatched with diagonal lines), a middle layer of concrete (stippled), and an outer layer of masonry (hatched with diagonal lines). The structure is shown in a plan view, with a rectangular section cut away to reveal the internal layers. Labels 'Masonry' and 'Concrete' point to their respective layers. A title box at the top right contains the text 'Plan of C. Iron Key'.

Fig. 1078.

It is illustrated in fig. 1078, and it will be at once observed what a solid structure will result from this method of design. It will be observed that the large blocks of masonry are set in circular form and a cast-iron key is provided at the crown, this being specially designed in accordance with requirements already set forth. In de-

signing any leap weir, however, the best thing to do first is to draw out to a large scale the actual path the water takes when flowing under the influence of gravity over a precipice, this path being also governed by the actual depth of water in the

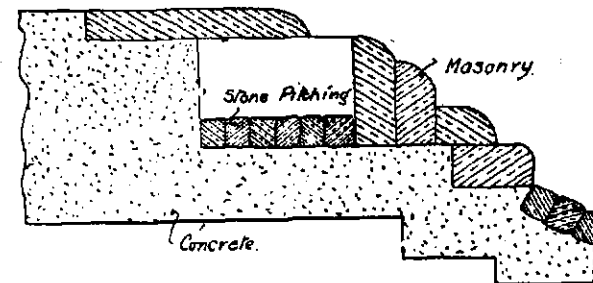


FIG. 1079.

channel leading to that precipice. The curve, as the reader should now be well aware, is parabolic, and by referring to fig. 1080 we shall understand the method of drawing out this curve.

h = depth of water in channel (effective, *vide* Hydraulics). x = proposed width of orifice. Set off $ad = \frac{g}{2}$ and $c'z = \frac{2}{3}\sqrt{gh}$, and draw a parallelogram as shown by the dotted lines, divide the two sides into nine equal parts as shown (or any number deemed necessary), joining up as shown. The points of intersection will give the points of the curve.

While

$$x = 9/16 \frac{y^2}{h} \quad . \quad . \quad (522)$$

When

$$y = 2/3 \sqrt{2gh} t \quad (523)$$

t being time interval in seconds.

Or again,

$$x = \frac{1}{2} g t^2. \quad (524)$$

Another form of leap weir employed on waterworks to a fairly large extent is shown in fig. 1078. Its construction, however, cannot be said to be as stable as that shown in fig. 1078.

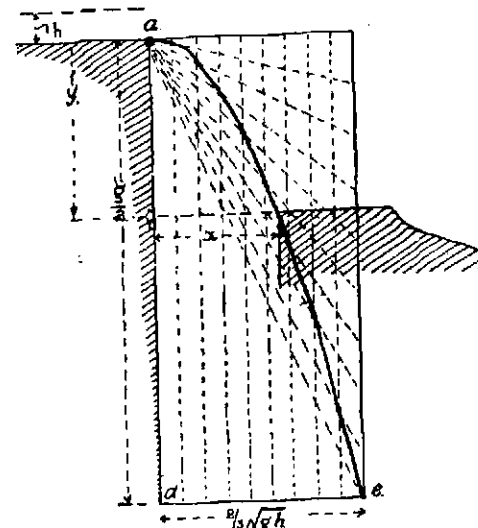


FIG. 1080

APPENDIX XVI.

THEORY of the module, shown: in figs. 524 and 525. See fig. 1081.

A is the adjustable sluice.

B is the fixed sluice or orifice.

The advantage of this weir for irrigation has already been set forth; in addition

we have the important fact that any change of level in the supply canal will not affect the ratio of the effective heads above A or B; hence the constant supply when once the orifice has been properly designed to suit the circumstances. This is proved as follows:—

$$Q = a_1 c_1 \times \sqrt{2gh_1} = a_2 c_2 \times \sqrt{2gh_2} \quad (525)$$

c_1 and c_2 are constants. a_1 = area of A. a_2 = area of B.

Hence $\frac{c_1 a_1}{c_2 a_2} = \sqrt{\frac{h_2}{h_1}}; \dots \quad (526)$

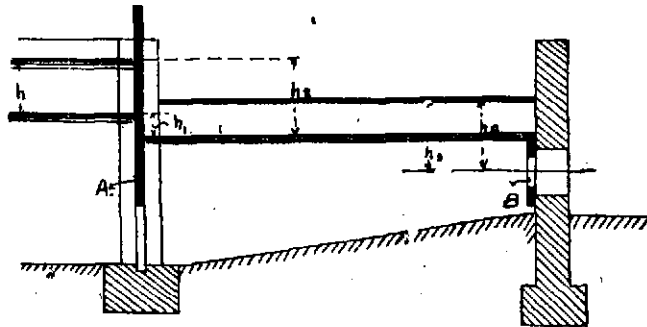


FIG. 1081.

and since the values of a_1 and a_2 are unchanged,

$$\frac{c_1 a_1}{c_2 a_2} = \sqrt{\frac{h_2}{h_1}} \therefore \frac{h_2}{h_1} = \frac{h_1}{h_2} \quad (527)$$

That is, the ratio of the effective heads remains unchanged.

APPENDIX XVII.

PRACTICAL method of laying out twist copings for bridge abutments (*vide* Chapter XXI.) and wing walls.

To set out ramp and twist, first draw the plan, then develop the line AB as shown, from which set up the outside elevation, finding a radius O to suit the curve required for the elevation. Having done this, mark off the number of stones required, and strike the joints from the radius O, then project down from these joints dotted lines as shown 1, 2, 3, 4, 5, 6, then transfer these distances to the plan, and draw the line radiating to the centre V. Now set up the inside elevation by developing the line $a'b'$. Having done this, transfer the distances 1, 2, 3, 4, 5, 6 at $a'b'$ in plan to the inside elevation, and project up by dotted lines, 1, 2, 3, 4, 5, 6, marking off the same height of each line as in the outside elevation. Through these points draw the line FF, which forms the curve of the inside elevation, then project down from bottom bed joints of outside elevation as shown by dotted lines a, b, c, d . Transfer these distances to plan the same as before, striking these lines to the centre V, then transfer them from plan $a'b'$ to the inside elevation, and project up as shown by dotted lines a, b, c, d , inside elevation, then get the thickness of the coping with the dividers, and mark off this distance from the curve FF, cutting the lines a from 2, b from 3, c from 4, and so on. By drawing lines from 2 to where a is cut and so on, we get the joints of the inside elevation; then draw the curve GG, and have the inside elevation finished, and also the inside face-moulds at 1, 2, 3, 4, 5, 6, having got the outside face-moulds before on outside elevation. To get the bed-

moulds for each stone from plan: No. 1 bed-mould, from B to 6; No. 2, 6 to d ; No. 3, 5 to c ; No. 4, 4 to b ; No. 5, 3 to a ; No. 6, from 2 to A. Now to work the stone No. 2, get the piece of stone, 1, 2, 3, 4, the thickness of the bed-mould, divide the face-moulds into equal number of parts at right angles to the lines, AB, $a'b'$,

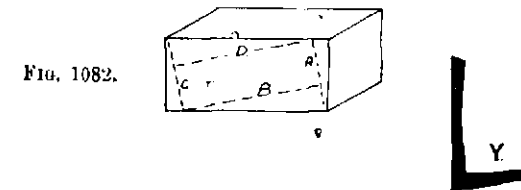


FIG. 1082.

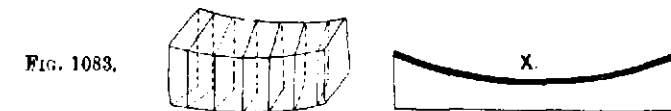


FIG. 1083.

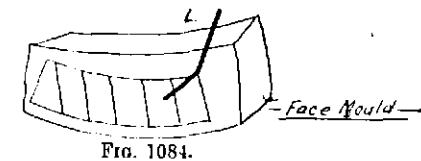


FIG. 1084.

as shown, then commence working the stone, fig. 1082, by working a draft at A, then, with the templet X, chisel the draft B, which will be the same sweep as the bed-mould, then twist the draft C with A at right angles to B, then work the draft

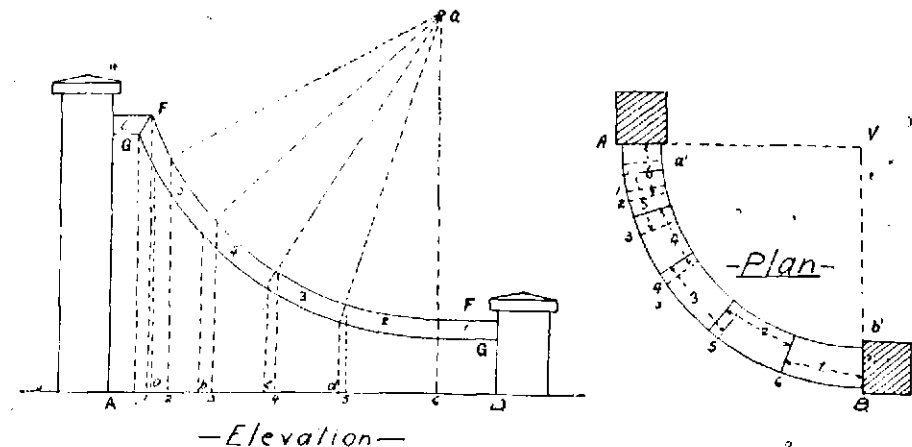


FIG. 1085.

FIG. 1086.

D to the templet X. Having done this, complete the face by working through BD. Work the inside face in the same manner to the thickness of the bed-mould, and to a templet the same sweep as the bed-mould at $a'b'$, then have a stone similar to fig. 1083. Now apply outside face-mould as at fig. 1084, then work the draft L to the bevel Y (which is cut to the joint of the bed-mould), this giving the radiating

away from the site, and the road is on a gradient of 1-400 and straight, and 74 chains long.

	s.	d.
1. Removing turf, rolling up same, and depositing, per cu. yd.	0	9
2. Surface excavation, maximum depth 18 in., including removing in barrows and depositing	1	0
3. Forming road to specification (No.), and levelling and forming path, per sq. yd.	0	2
4. Brick, hard core, 9 in. thick, for foundations of carriage-way, per cu. yd.	4	3
5. 4 in. of coarse gravel for carriage-way, per cu. yd.	5	4
6. 2½ in. gauge whinstone, installing 4½ in. thick, per cu. yd.	9	7
7. Footpaths, coarse gravel bed 4 in. thick, rolled, and limestone tar paving 2 in. thick, per sq. yd.	3	2
8. Killiney granite curb, dressed and cement-jointed, size 12 in. x 6½ in., per foot-run	1	7
9. Sett channel, grouted in cement, per foot-run	1	9
10. Cement concrete (5-1) bed for kerbstones and setts, per cu. yd.	12	6
11. Rolling, per sq. yd.	0	6½

The writer considers the work expensive, under the circumstances.

2. *Drains and Sewers.*—Pipe sewers, best extra-glazed fire-clay, cement grouted, laid on concrete bed, road surface made good, average excavation 10 ft. deep,

6 in.	10s. per yard.
9 "	12s. "
12 "	15s. "
15 "	18s. "
18 "	23s. "

Ordinary house drain, 4 in. diam., av. excavation 3 ft. deep, including above items, 6s. per yard.

Manholes, junctions, bends, etc., at cost price.

Circular sewers, 2 ft. 6 in. diam.	20s. per foot run?
" " 3 ft. 0 in. "	25s. " "
Egg-shape sewer, 4 ft. 0 in. x 2 ft. 9 in.	27s. " "
" " 5 ft. 0 in. x 3 ft. 0 in.	31s. " "

This includes a concrete backing, but not a blue brick invert, for which add 1s. to 1s. 6d. according to price of brick. This is considering excavation about 10 ft. in stiff ground.

Long lines of sewer in good ground, av. 7 ft. 6 in. deep, about 14s. per foot-run up to 5 ft. 0 in.

Sewer in tunnel, good ground, £20 to £30 per foot-run, including shafts and purchase of timber (not resold).

Manholes on house drains about 3 ft. deep, £5 each, in concrete. In stock brick, £3, including glazed channels and covers. For main sewers, £1 per ft. deep; under 6 ft. deep, £1, 5s.

In addition to the above, the following may be of interest.

The writer laid down 100 yds. of 18-in. sewer for £84, 10s., including two manholes, av. depth 6 ft. 9 in., while a length of egg-shape sewer in sandy soil, 3 ft. 6 in. x 2 ft. 3 in., half brick backed with concrete, 810 ft. long, including 4 manholes and one storm weir, cost just under £300 when the timber was resold. Special invert blocks were used.

Mr Dempsey, in his *Drainage Book* gives the following:—

Egg shaped sewer, av. 10 ft. deep,	
Size 4 ft. x 2 ft. 8 in.	10s.
system 3 ft. 6 in. x 2 ft. 4 in.	8s. 6d.
especially 3 ft. 0 in. x 2 ft. 0 in.	7s.
standard rate 3 ft. 6 in. x 1 ft. 8 in.	5s. 6d.
Hence the writer, 3 ft. 0 in. x 1 ft. 4 in.	4s. 6d.

per foot-run.
set in lime
mortar only.

The nature of the soil is not mentioned.

The same writer also records that in the main drainage of Torquay a sewer in tunnel known as the Waldon Hill Tunnel, in limestone rock, lined with 6 in. of concrete, and 1150 ft. long, was executed at £1, 16s. per ft.-run, the lining costing 2s. 8d. only, leaving £1, 10s. 8d. as cost of driving, or at the rate of £1, 19s. 2d. per cu. yd. of material removed.

3. *Sewage Works.*—See Chapter XXVIII.

Filter beds, brick and concrete walls and foundation, coke breeze filtering medium, 18s. to 30s. per sq. yd. If clay puddle walls only, 10s. per yd. Contact beds only, £1200 to £1400 per acre.

For disposal on sewage farms, about 25s. per head; or laying out 1 acre of land for broad irrigation, £110; intermittent filtration, £130.

Pipes laid out to sea, as in Chapter XXVII., not exceeding 2 ft. 6 in. diam. C.I. pipes, about £7 per yd.

Precipitation tanks, concrete walls, including appurtenances, excavating and banking up sides, £2 to £5 per 1000 gals. (open only). If covered, add 15s. per 1000 gals. The 1000 gals. = gallons capacity.

The writer has laid out in a fairly flat seaboard town a complete sewerage scheme. Population provided for, 21,000. Actual cost of works; tidal outfall, £54,000; = about £2, 10s. per head of population. No pumping required.

4. *Brickwork and Masonry* varies according to price of bricks and labour. The av. price of bricks may be set at 42s. per 1000 for comparison.

Taking the standard rod at 272 super feet of 14 in.	£16 per rod;
With ordinary lime mortar and rough joints, buildings	£16, 10s.
If in Portland cement mortar	£18, 10s.

Otherwise walls, for every ½ brick, 4½d.-5d. per sq. ft.

Taking 1 rod = 272 super ft. of wall 1½-bricks thick,—

If the work is in arches,

Rough	6½d. per sq. ft.
Gauged	2s.
Staff brick walling	1s.
Stone walls 18 in.-20 in. thick	1s. 5d.

provided local stone is used.

Bath stone, quarry face	2s. 7d. per cub. ft., set in lime mortar.
Portland stone, quarry face	3s. 3d. "
Ashlar, Bath stone	5s. "
" Portland stone	7s. 6d. "

Facing with ashlar only, 3s.-3s. 6d., according to quality of stone.

Granite walls, quarry face	6s. per cub. ft.
Ashlar	14s. "

Rubble masonry can be done for about 16s. per cub. yd., using local stone.

5. *Concrete Work.*—Foundations, P.C. concrete 6-1, £1 per cu. yd. if for ordinary buildings.

If for large engineering works (not subaqueous), 15s. per cu. yd.

Flooring, 6 in. deep, 3s. 3d. per sq. yd.; 6 in. deep, finished hard, 3s. 9d. per sq. yd.; 6 in. deep, finished with 1-in. neat cement or 1-1 sand and cement 4s. per super yd.

Concrete slabs for paving, 2 in. thick, 3s. 6d. per sq. yd.; 3 in. thick, 4s. 9d. per sq. yd. If laid, add 2s.

Rendering in cement, walls, etc., 1s. 9d. per sq. yd., ½ in. thick. For 1 mile ½ in. add 9d. to 1s., according to quantity.

Large engineering foundations, when under water, will be cheaper than in

joint of the stone; then square several drafts through, keeping the square on the upright lines, and on the radiating lines on the bed, as shown in fig. 1083; then, marking the upright lines on the inside, twist the inside face-mould in from draft 1, which will give the twist on the joints. Now work the stone through, and complete the piece of coping No. 2. (Remember to always keep the square on the upright lines on the face and to the radiating lines on the bed.) All the other stones are worked in the same way. Having worked the stone through clean, apply the section on the joints and trammel point from the face.

The plan and elevation are shown in figs. 1085 and 1086.

APPENDIX XVIII.

EXAMPLES OF COSTS OF ENGINEERING WORKS.

It has occurred to the writer that a few words on estimating and some examples of cost would be of value.

An engineer who is entrusted with design of works certainly benefits by a knowledge of quantities, but as the bills are often prepared by experts, only trifling mention has been made of them here. Ability to correctly draw up estimates for large works is gained only by experience. Again, nothing is more detrimental to an engineer's reputation than for his estimate to be considerably below the lowest tender, unless he is new to the particular district, or special circumstances are to be considered.

Most engineers of experience will possess a good knowledge of builders' prices. Again, a good price-book is very helpful in this direction. The difference in prices for work put in tenders is extraordinary. The writer has had some appalling examples before him, in spite of the fact that bills of quantities properly drawn up by a specialist were used; in fact, the practice of providing them in all, but the smallest jobs is strongly to be advised, although the time and labour, and hence expense, may be fairly large. The engineer's estimate is usually a much rougher one than those of the tendering contractors, principally because he has no time to spend going into details, and also because, after all is said and done, beyond a tolerably accurate idea of the reasonable cost of the work, his estimate has not much to do with the rest of the contract. There are various systems in vogue, some being quite original on the part of their users. Principally, however, the art of rough estimating resolves itself into one of three things:—

1. Taking the whole work at a certain rate per foot cube or yard cube, as compared with some similar existing work.
2. Finding with accuracy the actual cost of the brickwork or masonry, and taking a certain proportion of the cost of this for the other methods of construction (principally applicable to dwellings and public buildings rather than engineering works).
3. Taking rough quantities, which is by far the most satisfactory; and when the eye is well trained to this work, much accuracy will result.

For getting out estimates for work in a municipal engineer's office, however, the practice of taking it as so much per foot-run, foot or yard cube, foot superficial, etc., when the work is not of the greatest importance, is a very satisfactory way, provided the engineer has fairly accurate ideas of the cost of this class of work in his particular district. The estimate which he prepares is usually to satisfy a town council; and as such people, as far as the writer's experience goes, are never satisfied, and the works in question usually have to be done no matter what the cost is, hence the system, which is short and to the point, will be found very useful in practice, especially as the work is very often done by direct labour, and materials supplied at a standard rate for the time being.

Hence the writer has drawn up a list of probable costs of works which should

prove of value to the young engineer. Unfortunately, like the rest of engineers, he has not practised all over the three kingdoms, a fact obvious to all, and the reader is informed once for all that the cost of work will vary in different localities. The prices given are the result of about 25 years' experience in the city and county of Dublin, and may be taken as well up to the average.* Building materials are, of course, fairly high in cost, but labour is correspondingly cheap; at the same time, the average Irishman who gets 2s. 10d. per day will not do more than two-thirds of the work of an English or Scotch man at 4s. 3d. per day, even if he does as much. The skilled labour is of an inferior quality. Hence the writer is confident that under ordinary circumstances the following rough estimates will be very fairly accurate.

The prices are for finished work only, and include all charges.

1. Roads (vide also Chapter XIX.).

Country roads as per conditions and specifications given, about 4s. per foot super.

Per mile of 25 ft. wide	£2800-£3000.
„ „ 30 ft. „	£3500-£3750.

Light roads, 3s. per super ft.

Per mile	£1700, 25 ft. wide.
„ „	£2500, 30 „

No sewers, drains, or footpaths included.

Where these are included, and also sewers of moderate size, 6s. 9d. per super yard.

30-foot road	£5900 per mile.
35 „ „	£6130 „ „

Road maintenance (macadam).

Average traffic	£65	per mile per annum.
Light traffic	£35	„ „
Ordinary town traffic	£175-£225	„ „

Granite setts only cost £30-£50 per mile. Wood blocks, after first 2 years, £160 to £230 per mile. Asphalt (after first 2 years) £120 per mile, or about 4½d. per super yard.

Road rolling.—About 1s. 3d. per sq. yd., includes all charges, or about 9d. per cube yard of metal rolled in.

Watering roads, 14s. per mile, including water.

Pavements.—Asphalt, including 6 in. concrete bed, 12½ per square yd., 1½ in. thick.

For every additional ¼ in. add 2s. 6d., and for every ½ in. less deduct 5s.*

Asphalt alone costs about 6s. to 7s. per sq. yd. 1½ in. thick.

Ordinary concrete pavements, including a prepared foundation of broken brick or other suitable material, finished to a hard surface, checked and laid in bays about 5 ft. square, 4s. 3d. per sq. yd. Granite setts 7 in. x 3 in. laid on a properly prepared bed, and grouted as specified in Chapter XIX., 14s. 6d. to 15s. 6d. per yard super.

If 4 ft. x 4 x 4 cubes are used, deduct 2s.; where the actual setts are expensive the cost rises considerably.

Tar paving. When limestone is used (tar macadam), 5s. per square yard of 6 in. thick. Less thickness, slightly higher in proportion.

The average cost of streets having a 24-ft. carriage-way, and two 6-ft. footpaths, Telford foundation, steam rolled, macadam surface, concrete or tar paths, gutters 18 in., fireclay sewer, brick manholes and heavy covers, £3, 10s. to £4 per yard-run.

The writer has recently had tenders from a competent firm for the construction of a roadway 35 ft. wide, including two footpaths. The tender recommended acceptance has the following prices, which may be of interest. The railway mile

* At the present time, October 1910, buildings in Dublin are about 7 per cent cheaper than in the English provinces.

per cub. yd. This includes all caissons, air-locks, etc., and depreciation (on-hire) of the compressing machinery and general plant.

6. *Bridges (Masonry).*—To give an actual idea of this class of work is almost impossible, rough quantities usually being necessary.

The writer executed a single span road bridge under ordinary conditions for £1350; the length, including approaches, was 50 ft., and width between abutments 18 ft. This gives 900 super ft. = 1350 ÷ 900 = £1, 10s. per super ft.

Another bridge of three spans, 170 ft. long and 20 ft. wide, was executed for £9760, working out at nearly £3 per super ft. Masonry and labour both cheap in each case. These costs are, however, below the average. Steel bridges, under the same conditions, would appear to cost less by a quarter.

Swing bridges and machinery would cost as much as £6 per sq. ft.

7. *Land Drainage.*—(vide Chapter XXIII.) varies according to soil, depth of drains, type of drain, and distance apart.

Taking stiff clay drains, av. 20 ft. apart and 3 ft. deep, about £9 per acre, made up of pipes 31s. per 1000, £4, 5s., laying £3, and filling £1, 15s.

In ordinary loam this would be reduced to a total of about £6, while light soils would not cost more than £4, 10s.

Mr Dempsey, in his *Land Drainage*, gives much useful information regarding costs. Amongst other things it is stated that in a rectangular field 20 acres in extent, depth of drains 3 ft., distance apart 22 ft., the several costs would be as follows:—

Main drain—60 rods cutting and filling at 8½d.	£2 2 6
990 drain pipes at 40s. per 1000	1 19 7½
Minor drains—2261½ rods at 4½d.	42 8 0
Pipes, 37,320 at 30s. per 1000	55 19 7
Total cost	£102 9 8½

working out at £5, 2s. 6d. per acre.

For comparison, let us take the following:—

Acreage, 20 ac. 0 rd. 0 pl.

Drains, 4 ft. 6 in. deep and 45 ft. 0 in. apart.

60 rods of main drain cutting and filling at 1s. 2d.	£3 10 0
990 pipes at 40s. per 1000	1 19 7½
1109 rods of minor drains at 10d.	46 4 2
18,300 pipes at 30s. per 1000	27 9 0
Total cost	£79 2 9½

or nearly £3, 19s. 2d. per acre.

Considering stone drains, being built of round stones only, selected in size, the larger ones lowermost, 4 ft. deep, av. cost 2d. per yd. in light soil up to 4d. per yd. in stiff clay. Stone draining would cost in moderately light soil about £5 per acre, taking the drains only 2 ft. to 9 ft. deep, costing only a little over 1d. per yd., the drains being about 35 ft. apart. The total cost of draining peat moss may be set down as £1, 10s. 6d. per acre when the drains are 18 ft. apart and 3 ft. 6 in. deep. The principal cost here is the opening up—£1, 4s. 6d., the remainder being charged as outfalls, etc., and superintendence.

8. *Waterworks.* Mr Coleman, in his *Estimating*, gives that gravitation works cost £2, 10s. to £4 per head of population; in other words, £100,000 to £160,000 per million gallons supplied per day.

One of the most successful supplies in the whole kingdom is that of the city of Dublin, at Roundwood, costing £1, 18s. per head, which is considerably less than the Glasgow scheme at £3, Edinburgh at £3, 10s., Manchester at £2, 19s.

The cost of a storage reservoir about 50 acres in extent is stated by the late Mr Chas. Slagg, in his *Water Engineering*, as £20,000, made up roughly as follows:—

		Price per Unit Quantity.
12,000 sq. yd.	Clearing seat of embankment	£ 100
8,000 cub. yd.	Excavating puddle-trench	2,000
14,000 "	Clay puddle	1,750
74,000 "	Embankment	5,550
800 sq. yd.	Road over embankment	80
6,000 "	Pitching inner slope	600
4,900 "	Soiling outer slope	40
4,500 cub. yd.	Excavation for culvert and valve tower	900
4,000 "	" " by-wash	200
300 "	Concrete foundations	90
2,200 "	" " special m by-wash	770
700 "	Brickwork (culvert and valve tower)	840
500 "	Gravel puddle	100
1,500 cub. ft.	Dressed stone sills	150
	Valves	150
	Footbridge	80
	Gauge weir	100
	Total	£13,600
	Add contingencies	1,400
		£15,000
	Land and Fencing	5,000
	Grand total	£20,000

Conduits in tunnel cost on an average £5 to £6 per yd.

In pumping schemes, including the pumps, well-sinking, and reservoirs, and including interest and sinking fund, the cost of supplying water may be set down as 5d. to 9d. per 1000 gals., the usual water rate being about 1s. per 1000 by meter, and 11d. in the £ on the rateable value. These figures will vary considerably, however.

Wells.—Ordinary shallow wells below 30 ft. deep will vary in cost according to the diameter.

Taking a standard of 3 ft., the cost would be about 9s. per ft., to which add 3s. per extra ft. in diameter. Fractions of a ft. in proportion. This includes a half brick or plain 6-in. concrete sides. In wells of larger diameter than 5 ft. 6 in. one brick walls are necessary, or reinforced concrete, which will add 10s. to the cost of each ft. These figures assume ordinary soil.

For rock, boreholes are usually sunk, of comparatively small diameter, and lined with steel or copper tubes.

A hole 12 in. diameter, not deeper than 50 ft., would in hard rock cost about 28s. per ft.; for every 40 ft. extra depth add 1s. 6d. This excludes the lining, which runs about 25s. per ft. (or less).

Pumping Water.—The writer had under his charge a gas-driven pumping plant for a waterworks. Town's gas at 4s. per 1000 cube ft. was used. Av. calorific value, 640 B.T.U.'s, height pumped 210 ft. Gallons pumped per day (av.) 140,000 (nearly), including men's wages, oil waste, but not depreciation or interest, the day's pumping cost £1, 4s. 3d., or very nearly 2d. per 1000 gallons. The pumps during the year, in consideration were quite new. Producers are now installed, but not long enough for any valuable information to be at hand. The cost of this pumping is high. The cost of raising sewage, the lifts being usually comparatively low, averages under 1d. per 1000 gals., unless the pumps are very inefficient.

No attempt can be made to state with accuracy the average cost of pumping plants, so very varying are the tenders of this class of work.

The filter beds in a waterworks scheme cost a considerable portion of the whole works. Being usually not more than 6 feet deep, the area becomes large, and the walls are invariably of concrete; when uncovered they cost about £12,000 per acre, but when covered the cost is half as much again, though maintenance may be less. This will include the filtering material. Softening water costs usually under 1d. per 1000 gals. (*vide* also Chapter XXXIII.).

Water mains will vary in cost principally according to the cost of the pipes delivered, which varies with the diameter. The cost of opening up the ground, however, for a 2-in. water main is practically the same as for a 6-in. main, and then up to 12-in. there is no great increase. After this, however, especially if timbering is required, and also on account of the extra labour on the heavy pipes, the cost rises considerably.

The *av.* cost of mains, including coated pipes, lead joints, opening up ground, and watching, is—

Diameter of Main.	Cost of Pipes only, per yd.	Main as laid per yd.
2 in. and 3 in.	2s. to 2s. 6d.	£0 4 0
4 "	3s. 6d.	0 5 6
5 "	4s. 6d.	0 7 0
6 "	6s. 4d.	0 9 0
9 "	10s. 6d.	0 14 3
12 "	16s. 7d.	1 1 0
15 "	22s. 4d.	1 10 0
18 "	34s. 6d.	1 18 0
24 "	49s. 6d.	3 0 0

The service reservoir is another important item of cost. Great reduction of total cost results in large reservoirs. Those with a capacity of less than 100,000 gallons cost as much as £7 per 1000 gallons capacity, while those having a much larger capacity would be done for about £4, 10s., or even less. If the reservoirs are covered add £2 to £2, 10s. per 1000 gallons capacity. These costs assume concrete walls and asphalt waterproofing and soiling slopes. If blue brick lining is resorted to, add cost of same. Small underground storage tanks in concrete could be executed for about 1s. 3d. per cub. ft. capacity with 9-in. walls, which includes rendering, making good ground, and a manhole cover.

A cast-iron elevated tank would cost about 2s. per cub. ft. capacity, including supporting structure up to 10 ft., while a wrought-iron tank would cost about 3d. to 6d. more according to type of work specified.

9. *Refuse Collection, Disposal, and Destruction* (*vide* Chapter XXXI.).—In towns it will usually be found that on an average the amount of refuse removed per head per annum is $\frac{1}{4}$ ton, while in small places this will be considerably reduced. The cost of collecting such refuse will in large towns naturally present a higher figure because of the necessarily longer distance of removal. Where possible, the writer recommends the practice of establishing numerous small depôts for the carts to deliver during the day when the dust-bin system is in vogue, and removal in specially constructed waggons during the night by electric locomotive over the tramways system: this would be specially useful where the refuse is used to make up low-lying lands. Returning to actual costs, the cost of removal per head under ordinary conditions may be set down as 1s. per head per annum when dust-bins are supplied at cost price. In London the cost runs up to 1s. 4d. (about). To this must be added another 1s. 2d. for burning in a proper destructor, which includes labour, but not interest. The raising of steam and manufacture of the clinker into useful materials will all go to reducing this cost in large installations; in small ones a loss may even result when the interest on plant is paid.

In the city of Dublin, carting and disposal is stated at 4s. 8d. per ton.

The actual cost of destructor plant varies with the type. Messrs Heenan & Ergeude, in some particulars supplied to the writer, give the following figures:—

Levenshulme.—Cost of labour 7d. per ton, the men being paid at the rate of 30s. per week of 56½ hours.

Kings Norton.—4½d. per ton of refuse burned.

Redditch.—Electrical results.

Engines using 50 lbs. of steam per kw. hour generated 1050 units=77 units per ton of refuse.

A good plant, however, should not use more than 30 lbs. of steam per kw. hour, which brings this figure up to 129. 77 units supplied at 3½=451d.=£1, 17s. 7d., which is, of course, quite an outside figure.

Small destructors complete, including chimneys and buildings, appear to cost on an average £1500 per cell, the actual plant taking up about half of this only. Where the cells are as many as 8, the cost will fall to about £1150 per cell, while larger destructors still can be done for £950. Large seaboard towns dispose of their refuse, beyond risk of return, out at sea, for about 3s. per ton, including interest on the vessel.

MISCELLANEOUS ITEMS.

10. *Public Lavatories*, including w.c.'s, urinals, wash-basins, and attendants' rooms, about 4s. per cub. ft.

11. *Tramways (Electric)*.—Standard gauge per single track mile about £5500, which includes 100 lb. steel rails, fish-plates, bolts, nuts, copper, bonds. Excavation for and laying 6 in. concrete foundation. Laying rails, paving with setts to 18 in. on either side and between rails, and contingencies for diversion of water mains and house services.

Double tracks about double, less one-sixth. The trolley wires and supports, poles, etc., centre poles or bracket arms, £1500 per mile (double wires). Span wire one-third as much again.

12. *Street Subways*.

1. Foot passengers £20
2. Gas and water mains and cables only 6
3. Horse traffic 70 per ft.-run.

If under rivers, canals, etc., about double. Includes lining, etc., and approaches lighting, etc.

13. *Constructional Steel*.

- Joists, stanchions, etc. £15 per ton.
- Compound stanchions and built girders £18, 10s. per ton.
- Light road bridges, roofs, light steel building, etc. £25-£30 "

Roofs of very ordinary quality for rough work, span 30 ft., £5, 5s. per square, measured on slope or curve according to circumstances. For every extra foot span at 1s. per square.

First-class roofs for public buildings, skating rinks, etc., span 30 ft., £8, 10s. per square. Add 2s. for every extra foot span. Very heavy roofs for halls, railway stations, etc., over 100 foot span, the cost runs up to nearly double these figures, while very large works will cost as much as £30 to £35 per square.

14. *River Walls* (no extra difficulties).—21s. per yd.-cube, concrete only; 25s. per yd.-cube faced with concrete blocks; 30s. to 50s. per yd.-cube faced with granite.

15. *Chimneys*.

- Over 250 ft. £15 per ft. high.
- " 200 ft. and under 250 11 " "
- " 150 " " 200 10 " "
- " 100 " " 150 9 " "
- Under 100 ft. £7, 10s. to £8 per ft. high.

Steel chimneys about 20 per cent. less.

16. *Cofferdams* vary from £20 to £50 per foot-run under ordinary circumstances. Mr D. R. Clarke states that the great cofferdam at Grimsby Dock cost £29 per lineal foot, or £22 after the timber was resold. The same writer states that the Temple Pier dam, London, cost £17, 4s. 10d. per foot-run when the old material was sold.

17. *Ditches and Embankments* for land drainage slopes, 1 to 1, depth 2 ft., about 2½d. per ft.-run.

18. *Groynes*.—Light construction, 6 in. diam. round larch piles, 2½-in. boarding 2 ft. 6 in. deep, piles 6 ft. 6 in. centre to centre, about £1 per ft. run, according to price of timber.

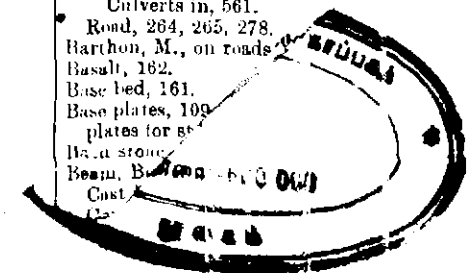
A good substantial groyne to following specification—

Piles, 12 in. x 12 in., creosoted, secured by 10 in. x 10 in. struts, 6 in. sheeting 3 ft. deep, secured by iron bolts with nuts and washers, about £2 per ft.-run; piles 10 ft. c. to c., and driven 9 ft. 6 in. into sand.

If old rails are used as uprights, costing £5, 10s. per ton, add 7s. per ft., which is easily saved in reduced maintenance and longer life of piles.

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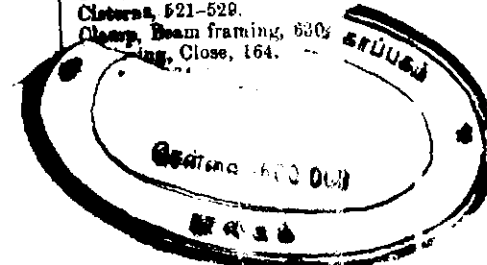
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