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Paper No. 200

“ASPHALT PAVING MIXTURES—THEIR DESIGN AND COMPOSITION”

By

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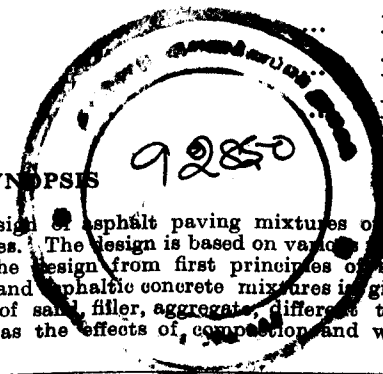
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SYNOPSIS

The Paper deals with the design of asphalt paving mixtures on the analogy of soil and cement mixtures. The design is based on various physical tests which are described. The design from first principles of sheet asphalt, stone filled sheet asphalt and asphaltic concrete mixtures is given. The effects of varying proportions of sand, filler, aggregate, different types and quantities of bitumen as well as the effects of compaction and water absorption are also demonstrated.

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1. INTRODUCTION

1.1. Asphalt paving mixture design receives only cursory treatment in text-books and in technical educational institutions. Engineers may be presented with many formulae or recipes, arranged in the guise of specifications, some with limits set so wide that they include mixtures both good and bad, some so precise that they may seldom be achieved with economy. They will be fortunate if they are given any general principles of design, principles which must control the design of asphalt paving mixtures in countries where long experience of the success of mixture formulae under the conditions obtaining are lacking, principles, therefore, that an Indian engineer must know.

1.2. This Paper is written with the object of enunciating the principles of asphalt paving mixture design, in the hope that younger engineers will see in the subject some scope for research and that the more senior ones will be aware of the difficulties which confront them when they draft their specifications for hot mixed asphalt surfacings.

1.3. By "asphalt paving mixture" is meant a material composed of bitumen and mineral matter prepared by mixing in a heated condition bitumen, which may either be a natural product, such as Trinidad bitumen, or the residual product of the distillation of petroleum oil, with sand, stone chippings and fine powder, the latter generally being known as filler. Whilst much of what is written in this Paper may apply to tar products, this is not necessarily so and only mixtures as defined above are now being considered.

1.4. The change in dry density which occurs with variation in moisture content is shown in diagrammatic form in the case of soils by plotting dry density against moisture content. This method of presentation, although satisfactory for soils, does not indicate with sufficient clarity the changes in asphalt mixture characteristics which follow the variation in the mixture composition. In the upper portion of Fig. 1 are shown typical curves of the dry density-moisture content relationship of a soil compacted to two different standards. Lines across the chart indicate the dry density : moisture content relationship of saturated soils and of those containing 5 per cent and 10 per cent moisture. Such diagrams are familiar to all engineers who have studied soil mechanics.

1.5. In the lower portion of Fig. 1 the same dry density curves are plotted, although to a reduced vertical scale, but

curves are introduced showing the voids in the soil and the volume of these voids which are occupied by moisture. The difference between the ordinates of the two curves for any given moisture content indicates the air voids. To complete the picture, the California Bearing Ratio of the soil at various moisture contents is shown. The same method of presenting test results can be used in reporting tests on concrete mixtures in which an increasing proportion of water-cement paste is used having a specified water-cement ratio. It would surprise many concrete engineers to know that the same falling off in strength of concrete, similar to the falling of in C. B. R. in the case of soils, can be observed as the volume of water-cement paste increases.

1.6. The Author considers that this method of presentation of results in the case of asphalt paving mixtures is as valuable to the engineer as an X-ray photograph is to a surgeon. The changes in characteristics can be noted at each stage as the bitumen content is increased. The comparative results obtained with different aggregates or different combinations of aggregates can be studied and final decisions on composition can be made. This is what is covered by the term "design" in reference to asphalt paving mixtures.

1.7. The term "voids" is, in want of a better term, somewhat loosely used in asphalt terminology. "Voids in the aggregate" simply means space not occupied by aggregate. These voids may be occupied by air or by bitumen. Similarly "voids in sand and stone" might be occupied by filler, bitumen or air. If the sample is subjected to a water absorption test, the air voids may become partly or entirely filled with water.

It will be apposite to stress here that the percentage of voids in sand or stone samples is not a fixed characteristic but varies with compactive effort and lubrication. Descriptions are often given in text-books² of methods of determining voids in sand and stone by filling the voids with kerosene or water. Such efforts are usually a complete waste of time as the voids thus determined do not approximate to the voids in a compacted and lubricated mass. The voids referred to in this Paper are invariably determined from the weight of a compacted sample composed of materials in known proportions and of known specific gravities.

2. TYPES OF ASPHALT PAVING MIXTURES

2.1. There are two distinct types of asphalt paving mixtures, (a) Open graded and (b) Dense.

2.2. Open Graded Mixtures

One open graded type of asphalt mixture is well known in India and is termed "premix". This is simply a mixture of sun-warmed stone with hot bitumen, a small percentage of kerosene being added to the mix to assist the distribution of the bitumen. In this case, the working conditions dictate the design. Fine material is excluded from the mix because cold fine material cannot be coated with a viscous fluid and the addition of large quantities of kerosene to reduce the viscosity would be expensive and might necessitate delaying the use of the mixture until some of the kerosene had evaporated.

The quantity of bitumen used is normally that found in practice to give satisfactory results and the optimum bitumen content bears no relation to the optimum bitumen content which might be given by the results of tests on a carefully graded dense mixture. The air voids in the stone are probably from 30 to 40 per cent and the quantity of bitumen used, usually from $3\frac{1}{2}$ to 4 per cent by weight or 9 to 10 per cent by mix volume, is far from being sufficient to give a waterproof mixture. Open graded mixtures owe their stability to the internal friction of the aggregate and the requisite bitumen content is related to the surface area of the stone. Such mixtures are not confined to premixed material using sun-warmed aggregate and, when such mixtures are prepared using heated stone, filler, or fine material passing 200 mesh, may with advantage be introduced; in this case the material coating the stone may be considered as being a mixture of filler and bitumen and the quantity required is still related to the surface area of the stone. The filler is used with the object of stiffening the bitumen and preventing it from running off the stone. The use of a suitable filler such as cement or limestone dust will also assist in securing adhesion of the bitumen to the stone in the presence of water.

Since open graded mixtures are porous, their life will be greatly prolonged if the sub-surface is also porous so that water is not held for long periods in the interstices of the mixture. Although traffic may give a superficial polish to the surface, particularly if a small stone is used, premixed surface, unless provided with a seal coat, should never be considered as giving a high degree of weather protection to the underlying macadam.

It should also be remembered that an asphalt surfacing of any type restricts evaporation from the underlying macadam and if the water-table is unduly high, the moisture content below the asphalt surface will be higher than it would be were no surfacing provided. This condition can be dangerous if the binding material in the macadam is plastic.

2.3. Dense Asphalt Paving Mixtures

Although "premix" has an undoubtedly wide scope, it cannot have the life of a properly designed dense, waterproof asphalt mixture, which should give trouble-free service for at least fifteen years.

A dense asphalt paving mixture is composed of stone, sand, filler and bitumen, stone being omitted in the case of sand sheet asphalts. The first asphalt pavings were prepared from naturally impregnated limestone rock but early in the 20th century mixtures of Trinidad bitumen, a natural product, with sand and stone were used for road surfacing in the United States and in England. With the growth of the oil industry, refineries were faced with the problem of disposing of the residual bitumen and the increasing demand for smooth roads stimulated research. In 1905 Clifford Richardson published "The Modern Asphalt Pavement"; this remained a standard reference work for twenty years. Richardson formulated a "standard grading" for sand asphalts based on analysis of a large number of successful pavements. This is given below:

Bitumen	...	...	10.5	per cent
Passing 200 mesh	...	...	13.0	"
Passing 80 mesh retained 200 mesh			26.0	"
Passing 40 mesh retained 80 mesh			34.5	"
Passing 10 mesh retained 40 mesh			16.0	"
			<u>100.0</u>	

2.4. In 1924 Moray F. Macnaughton, at the Tenth Annual Conference on Highway Engineering, gave the results of investigations carried out to find the effect of varying the quantity of filler, or fine material passing 200 mesh, in sand asphalts. He explained that an increase in filler content led to a noticeable excess of bitumen and a consequent softening of the mixture. Until that time it had generally been considered that the high surface area of the filler stiffened the fluid bitumen and gave stability to the mixture. Whilst this effect is evident, it was from that time appreciated by many engineers that an increase in the filler increased the proportion of filler-bitumen lubricant in the composition and if the quantity of filler-bitumen exceeded the space available in the compressed sand, the mixture would be rich and unstable. The identical effect when water in compressed soil is present beyond the optimum moisture content is, of course, now widely known, and alternative methods of presenting test results have been shown in Fig. 1. Fig. 2 shows the results which may be expected when successive increments of bitumen are added to an asphalt mixture.

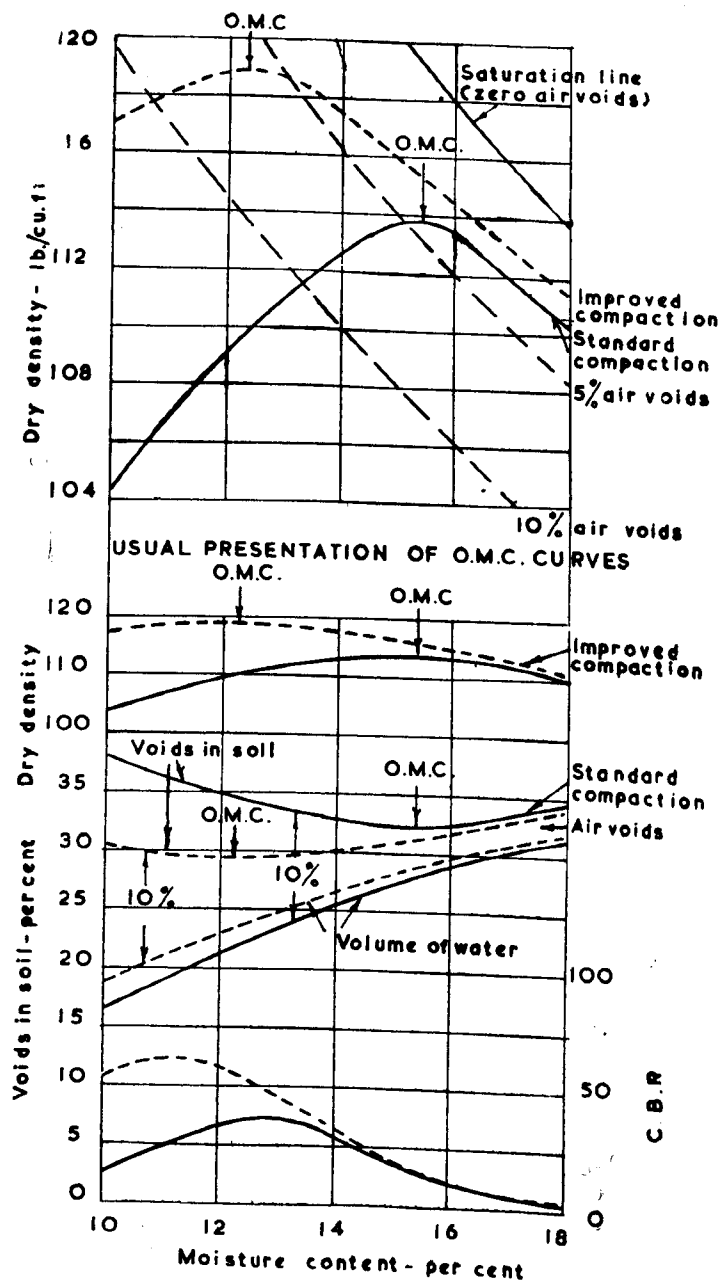


Fig. 1. Improved presentation of soil characteristics

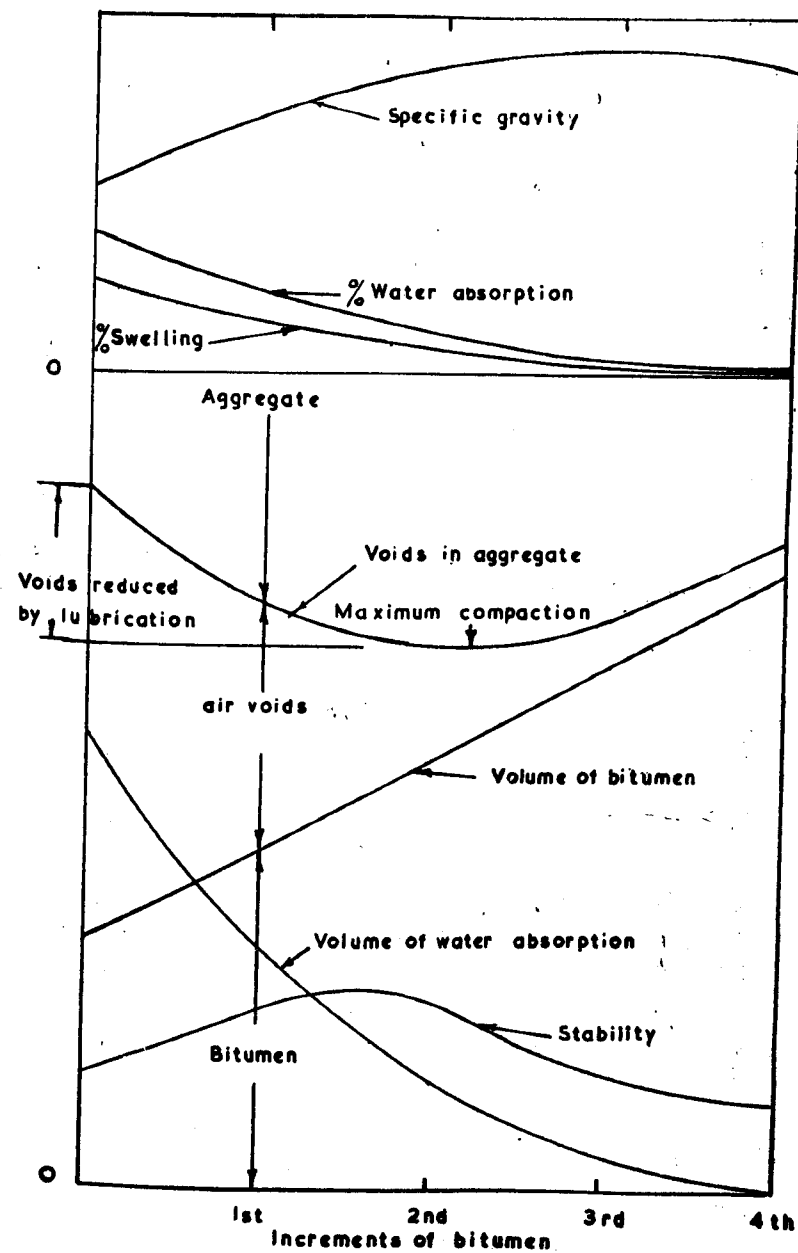


Fig. 2. Asphalt mixture characteristics

2.5. The initial increase in bitumen results in a reduction in the air voids, partly by filling them with bitumen and partly by lubricating the mass and assisting the particles to come closer together. This result is reflected in the increase in density as indicated by the specific gravity and by the increase in the strength of the mixture as indicated by the stability value. The measurement of stability will be dealt with later.

The second increase results in a continuation of the same effect, but the third increase results in a marked change in the characteristics. The air voids are only slightly reduced; the particles of aggregate no longer come closer together but are pushed further apart; the specific gravity shows no increase and the stability is considerably lowered. These effects are generally continued with a further increase in bitumen. The bitumen is now present in such a volume that intimate contact between the particles is impossible; its lubricating effect is such that internal friction no longer contributes to the strength of the mixture which now depends on the viscosity of the bitumen. At this stage the stability curve tends to flatten.

2.6. The space in the mixture occupied neither by aggregate nor bitumen is shown in the diagram as being occupied by air. Although water is obviously not present immediately after the compaction of the mixture, it may enter after the mixture has been laid on the road. The amounts shown in the diagram, being approximately equal to the air voids, can be introduced into samples of many mixtures by subjecting them to a vacuum followed by the admission of water under pressure. By this method water can be introduced into an asphalt mixture in a few minutes. On the road the entry of water is a time effect and although bitumen is so often considered as a waterproofing medium, water is one of the principal destructive agents which asphalt paving mixtures must be designed to resist. It will be noticed that the absorption of water is accompanied by swelling. If limits to the various characteristics are specified, such limits being based on knowledge of the conditions of climate and traffic to which the asphalt mixture will be subjected, a composition complying with the specification can be chosen from the diagram or the materials used in compiling the diagram may be rejected if no composition shown is satisfactory.

3. MATERIALS USED IN THE MIXTURE

3.1. Bitumen

It would be out of place here to give a full description of bitumen and its properties. The grades used for paving are 30 - 40,

60 - 70, and 80 - 100 penetration. It should always be remembered that these penetrations are measured at 25°C. and bitumens from different sources having the same penetrations at 25°C. may not have the same viscosity at 60°C., a temperature often encountered on exposed black surfaces in India.

3.2. Filler

The fine material known as filler used in an asphalt paving mixture has two functions. Firstly it fills the voids in the sand. Secondly it stiffens the bitumen and assists in maintaining the stability of the mixture after the point of maximum density has been reached and the mixture can no longer rely on the internal friction of the mass.

To fill the voids in the sand, the filler must be fine and the British Standard specifies that 85 per cent should pass the No. 200 mesh sieve. The effects of the fineness of grinding and of the characteristics of the filler on its stabilising or stiffening properties have been dealt with fully elsewhere by the Author³. Fig. 3 shows the rise in melting point in °C. of a bitumen having a melting point of 54°C. due to the admixture of various proportions and kinds of filler. The mixtures are by volume as the fillers have differing specific gravities. The proportion most commonly used in practice is an approximately equal quantity by weight or 40 per cent of filler by volume but it will be shown later that some fillers can with advantage be used up to 70 per cent of the bitumen by volume.

The effect of the filler is complex in this respect. If the filler has a high stabilizing effect on the bitumen, the filler-bitumen mixture will have a lower lubricating effect on the rest of the mineral matter and a larger volume of bitumen will be required to fill the voids. If the mixtures are designed to give a desired stability and voids, this effect on the optimum bitumen quantity will be taken care of but a mixture grading which does not specify the type of filler is obviously incomplete.

The property a filler must have if it is to be used with confidence is resistance to stripping in the presence of water. Probably the most satisfactory filler in this respect is cement but limestone dust is good and certainly much better than granite dust or dust produced from any igneous rock. The latter fillers can be used if the mix is very rich and the water absorption is low but failures have occurred which can be attributed to the use of a filler which permits the stripping of its coat of bitumen in the presence of water.

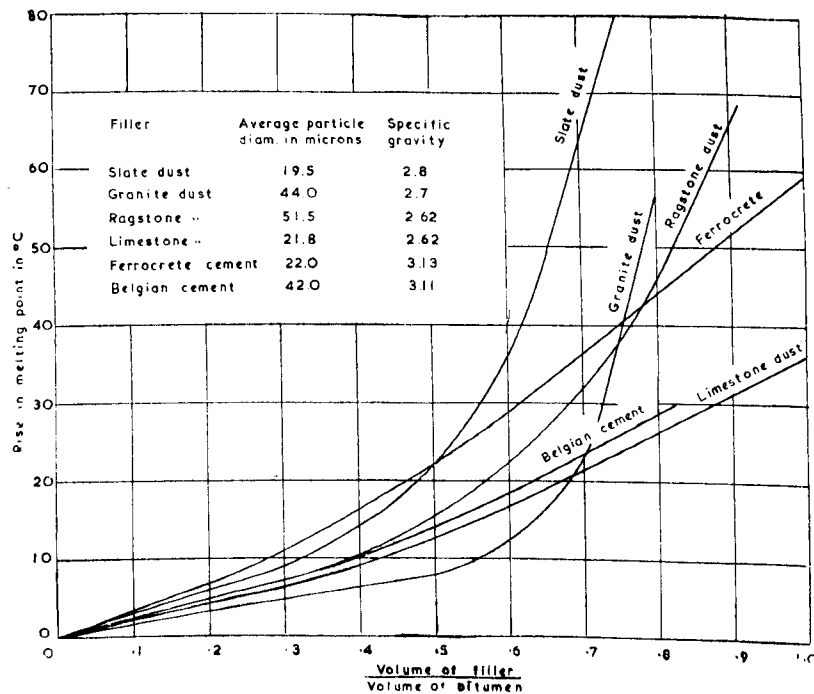


Fig. 3.

3.3. Sand

The first asphalts were prepared as synthetic substitutes for natural limestone rock asphalt which was ground to a powder form. Sand was the obvious choice of a material for adding to bitumen. It has already been shown that Clifford Richardson arrived at his standard grading from analysis of successful sand asphalt carpets. British Standard 594 : 1950⁺ for Rolled Asphalt (Hot process) specifies a sand which includes Richardson's grading as will be seen from the particulars given below :

			British Standard		Richardson
			Min.	Max.	son
			per cent	per cent	per cent
Total passing No.	7 B. S. sieve		95	100	100
" "	No. 25 B. S. sieve		75	98	93
" "	No. 72 B. S. sieve		15	58	45
" "	No. 200		0	3	0

Although the British Standard sand grading appears to be based on that recommended by Richardson, considerable latitude is allowed. Whether a restriction of limits to such fine sands is desirable or not will be discussed later.

3.4. Stone

Stone was first added to an asphalt paving mixture with a view to cheapening the mix, and the term stone filled asphalt came into being. Such mixtures, which contain upto 25 per cent of stone, have many of the characteristics of a sand asphalt; the stone serves no structural purpose but renders the mix cheaper in that each particle of stone in a fully consolidated mixture replaces an equal volume of sand asphalt.

Both sand carpets and stone filled asphalts are laid on a binder course of bitumen coated stone of an open texture which is intended to form a bond between the surfacing and the foundation. A binder course may often be used to regulate an uneven foundation before a more expensive wearing course is laid.

The hardness of the stone is seldom an important factor in the wearing properties of the finished surface and igneous, calcareous, silicious rocks and crushed gravel have been successfully used. To ensure that the stone plays its proper role in the structure it is usually specified that the largest stone shall be not more than one half nor less than one third of the final thickness of the consolidated course. Trials carried out in England appear to show that the grading of the stone is not critical⁶ but it must be remembered that the grading does affect the voids in the stone, and in a mixture having a high stone content the grading will determine the optimum bitumen content.

4. PROPERTIES OF THE COMPACTED MIXTURES

4.1. Some specifications for asphalt paving mixtures are prepared from the knowledge gained by long experience of the behaviour of mixtures of known composition used under known conditions. Were all structures designed by such conservative methods, engineering progress would be extremely slow. It was shown earlier that asphalt paving mixtures, soils and concrete had much in common and the design of the three types of mixtures is capable of a common approach.

As a preliminary to design, we must postulate the required properties of the mixture. These include:

- Wear resistance
- Deformation resistance
- Water resistance
- Economy

4.2. Wear Resistance

Dense asphalt mixtures, except those having a very high stone content are highly resistant to abrasion. For wear to take place there must be some abrasion followed by the removal of the abraded material. In the case of asphalt, the surface tends to yield to accommodate a high intensity of point or line loading. Thus the edge of a bullock cart wheel or a steel trolley wheel may make a slight indentation in the asphalt surface but there is no abrasion and unless the load is repeatedly placed on the same point there is no permanent damage. To ensure resistance to wear, the bitumen content must be sufficient to allow the mixture to yield under high loading and the consistency of the bitumen must be such that the mixture can yield without cracking.

4.3. Deformation Resistance

The determination of the degree of yielding of an asphalt mixture under varying loads and conditions has occupied the minds of asphalt technologists from the earliest days of asphalt construction.

In 1924 Macnaughton used an indentation test in his studies of the effect of increasing the filler content of a sand sheet asphalt. By 1929 the Hubbard field stability test was becoming widely used in America. In this test the load required to force a compressed sample of the mixture through an orifice at a standard rate was measured. In 1932 the Author described a test which involved loading the central area of a $4\frac{1}{2}$ in. diam. disc using a standard rate of loading³. He also adopted the Hubbard Field test by measuring the limit of proportionality between deformation and load.

4.4. In England the Road Research Laboratory studied all methods of stability testing but could not recommend any as a standard as none measured any basic property of the material.

Recently the search for a suitable test was continued⁶. Eight tests were investigated, these being:

- Constant load tests at 45°C.
 - Road machine (dynamic loading)
 - Wheel tracking (dynamic loading)
 - Indentation test (static loading)
- Constant rate of strain tests at 45°C.
 - Marshall test
 - Hubbard-Field test
 - Unconfined compression test
 - Hveem stabilometer
- Equilibrium test at 30°C.
 - Triaxial test

This investigation shows that while all the testing methods differentiate between mixtures having varying bitumen contents, the results do not always agree with those indicated by observed behaviour on the road. One result of considerable importance is that tests which employ a low rate of imposed deformation did not show the large increase in stability on increase in viscosity of the bitumen that would be expected from a knowledge of the road behaviour of asphalts. For compositions best suited to conditions in the United Kingdom it is suggested that the relative assessment of stability is best placed in dynamic loading tests employing pressures equal to those experienced in the road. Such tests are the Road Machine and the Wheel Tracking tests. These tests do attempt to reproduce road conditions in the laboratory but are of little assistance in research or routine testing as the duration of the test may be weeks. The Hubbard-Field, the Marshall and the Taylor tests give results in minutes and are very valuable guides to probable behaviour on the road so long as the results obtained are viewed in the light of other physical tests.

4.5. Water Resistance

Water has been mentioned as being one of the most destructive agencies operating on asphalt roads. In a premix, the bitumen film is comparatively thick but the film in dense asphalt paving mixture is thin and in the presence of water its adhesion to the mineral matter may break down and the mixture would then rapidly disintegrate. Whilst the correct choice of filler

will reduce the risk, the mixture must be designed so that the bitumen is in the Design Range and the voids have been reduced as far as possible without affecting adversely the stability. In the United States, it is usually specified that the voids in a thoroughly compacted mixture should not be more than 5 per cent nor less than 3 per cent. For routine testing, the calculation of voids with a view to ascertaining the liability to absorb water is far too long a procedure and the vacuum method mentioned earlier can be used to advantage. The simple apparatus required is shown in Fig. 4. It will be shown later that water absorption is invariably accompanied by swelling. The Author is of the opinion that the degree of swelling is of primary importance as an indication of the suitability of a mixture for resisting the effects of water. It is interesting to note that voids, water absorption and swelling are not characteristics of a mixture brought under control by British Standard 594 : 1950.

4.6. Economy

True economy normally follows from the use of locally available aggregate. Bitumen and filler may have to be imported but a skilled designer will be able to use local materials to the best advantage. It is the nature of the locally available materials which often decides the composition of the paving. There can be few engineering products which allow the designer as much scope in the use of a variety of raw materials as does an asphalt mixture. To quote from one hand-book on this subject, "The very heart of bituminous construction is the adaptability of methods and materials to local conditions."⁷

4.7. The principles of asphalt paving mixture design have been considered and the application of these principles to practical design can best be followed by the actual design of mixtures for given conditions using materials available.

5. MEASUREMENTS OF PHYSICAL CHARACTERISTICS OF MIXTURES

5.1. The method proposed by the Author for designing asphalt paving mixtures consists of making a series of mixtures with an increasing bitumen content, measuring the properties of each mixture in the series and then deciding on a bitumen proportion which will give, with the aggregate selected for testing, a mixture having properties within the limits fixed by experience.

5.2. Characteristics to be Measured

For every combination of bitumen and aggregate the following characteristics must be measured :

- (a) Specific gravity of dry sample.
- (b) Water absorption, rapid method.
- (c) Swelling, rapid method.
- (d) Air voids in dry sample.
- (e) Stability.
- (f) Resistance to deformation.

5.3. Preparation of Test Samples

The sample required for the Taylor stability test is $4\frac{1}{2}$ in. in diameter and about $1\frac{1}{4}$ in. thick. Such samples can very conveniently be used for making the water absorption and swelling determinations. From 1000 to 1150 gms. of mixture are prepared and heated to a temperature of 160°C. and the mixture is compacted in a mould using 25 blows of the standard Proctor tamper as used in soil mechanics. The sample is then reversed and 25 blows are given on the other side. The degree of compaction obtained is comparable with that obtained with modern machinery on the roads.

5.4. Specific Gravity of the Sample

The specific gravity may be found by dividing 'the weight of the sample in grammes' by 'the volume of the sample in c.c.s as found by micrometer measurements.' A more precise method is the measurement of the displacement in water at 20°C. or in water of some other known temperature, a correction in the latter case being applied. The weighing in water of porous samples having a low bitumen content must be done quickly before absorbed water increases the weight.

5.5. Water Absorption—Rapid Method

For the rapid determination of water absorption the rapid method already mentioned is used. Two of the standard test pieces can be inserted in the apparatus shown in Fig. 4 (Photo) where they are subjected to a vacuum of 25 in. of mercury for 10 minutes. Water is then drawn into the apparatus and air pressure of 50 p.s.i. is applied and maintained for 30 minutes. The samples are then removed and weighed in air and in water. From these weighings both the water absorption and the swelling of the sample can be determined,

The water absorption determined by the rapid method may not, in the case of very absorbent mixtures, equal that found after 24 hours soaking in water but the use of this method during preliminary work gives results with which routine control tests can be checked.

5.6. Air Voids in Dry Sample

Most specifications fix the limits for air voids in a compacted mixture. If the exact composition of the mixture is known—as it must be in the case of a freshly prepared sample—and the specific gravities of the materials used are known, the air voids can be found by deducting from the measured volume of the sample, the volumes of each of the materials of which it is composed. An approximate determination of air voids can also be made by ascertaining the apparent specific gravity of the loose, uncompressed, freshly mixed material and comparing this with the apparent specific gravity of a compressed sample.

5.7. Filling of Air Voids by Absorbed Water

If W_1 = original dry weight of the sample

W_2 = original weight in water—the weights being in grammes

then $W_1 - W_2$ = original volume in c.cs

If W_3 = weight with absorbed water

W_4 = weight „ „ „ in water

then $W_3 - W_4$ = new volume in c.cs

$(W_3 - W_4) - (W_1 - W_2)$ = increase in volume, swelling = S

$W_3 - W_1$ = increase in weight, absorption = A

$W_4 - W_2$ = increased weight in water = W_s

Since $(W_3 - W_4) - (W_1 - W_2) = S$

$(W_3 - W_1) - (W_4 - W_2) = S$

therefore $A = S + W_s$

It is, therefore, clear that the water absorbed can be divided into that portion which is represented by the swelling and that portion which has entered the voids in the original dry sample. As a percentage of the original volume, the water displacing air in the sample may be expressed thus:

$$\frac{W_4 - W_2}{W_1 - W_2} \times 100$$

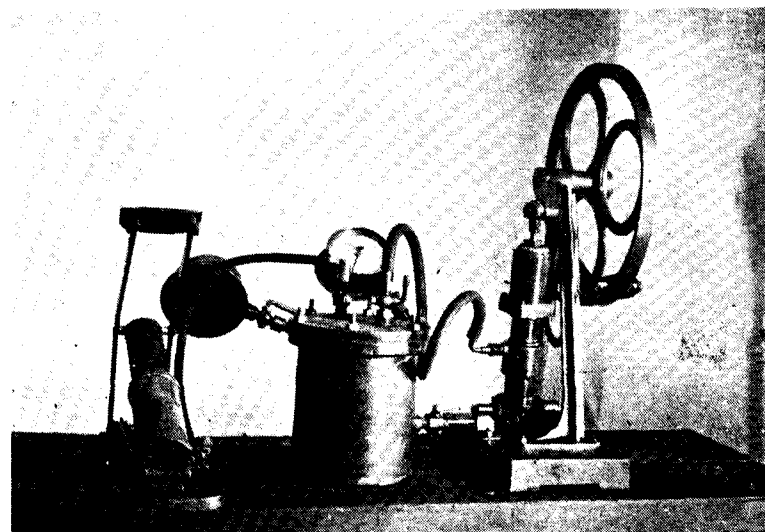


Fig. 4.

Water Absorption Testing Apparatus.

It will be shown later how close this figure can be to the air voids determined in the usual way.

5.8. Stability

Comparative tests recently carried out by the Roads Research Laboratory, Harmondsworth, showed that dynamic tests were of more value in determining comparative stabilities than were constant-rate-of-strain tests. Unfortunately dynamic tests take a long time to give a result and a test which is to serve both as a routine control as well as an aid to research must give an immediate result. Modern mixing plants produce 50 to 100 tons of mixed material per hour and a test result available even in 24 hours is nothing more than a confirmation or otherwise of compliance with the specification. Such tests are of little assistance as controls.

In the Taylor stability test, the sample is tested at the sun temperature of the area in which the mixture is to be laid. In India this is frequently 60° C. To the heated sample, is applied a central load increasing at the rate of 150 p.s.i. per minute through a plunger 2 inches in diameter. The deformation is read by means of a dial gauge and readings are continued to 0.2 inch or failure. A mathematical analysis of the stresses set up during the test has been made by Swaminathan⁸ but the point of great interest is that deformation takes place even in the toughest samples of asphalt at a rate proportional to the load. A very small proportion of the deformation is elastic but is sufficiently small to be ignored. Failure occurs when the annular ring surrounding the directly stressed area fails and the pressure at failure is taken as the stability value. If no failure as described occurs, the stability value is taken as being the pressure required to give a deformation of 0.2 inch. The Taylor Stability Testing Machine is shown in Fig 23 (Photo) while Fig. 24 (Photo) shows a test sample before and after testing.

Such tests may not predict exactly how a mixture will behave on the road but they certainly serve as a guide during experimental work and as controls on mixtures produced at the plant. No engineer who has once adopted some form of stability test would ever again dare to lay untested mixtures.

5.9. Resistance to Deformation

Road surfaces are seldom entirely destroyed by deforming forces and it is the resistance of mixtures to comparatively low pressures which is of interest. The Author takes as Resistance to Deformation the increase in pressure required to cause a deformation of 0.01 inch. Normally, the resistance to deformation

is proportional to the stability but some mixtures may break suddenly while others having the same resistance may remain unbroken at 0.2 in. deformation and have a much higher stability. Such results are indicative of a desirable toughness.

6. ASPHALT MIXTURES FOR A LARGE TOWN

6.1. Let us assume that asphalt mixtures are required for the streets of a large town with heavy traffic and that the following conditions apply.

Weather—heavy monsoon for 4 months.
maximum sun temperature 60° C.

Materials

Bitumen 30/40, 60/70, 80/100, 180/200
penetration.

Filler—Limestone and basaltic stone dust.

Sand—Coarse sand No. 1

Fine sand No. 2

Stone—Basalt as required.

This town is fortunate in that there is no lack of materials and the problem is to find the most satisfactory combination. The wide range of available materials enables all types of mixtures to be investigated.

6.2. Choice of Sand or Sand Combinations

The sands available have the following gradings:

	Sand No. 1 per cent	Sand No. 2 per cent
Passing 7	100	100
„ 14	72	99.2
„ 25	51.2	97.6
„ 52	30.7	76.3
„ 100	5.1	28.9
„ 200	0.7	2.3

In Fig. 5 these gradings are plotted and for comparison are also shown (a) the Fuller curve for material passing 7 mesh, together with (b) the Fuller curve corrected to give the sand grading and omitting material passing the 200 mesh sieve (c) the sand grading recommended by Clifford Richardson and (d)

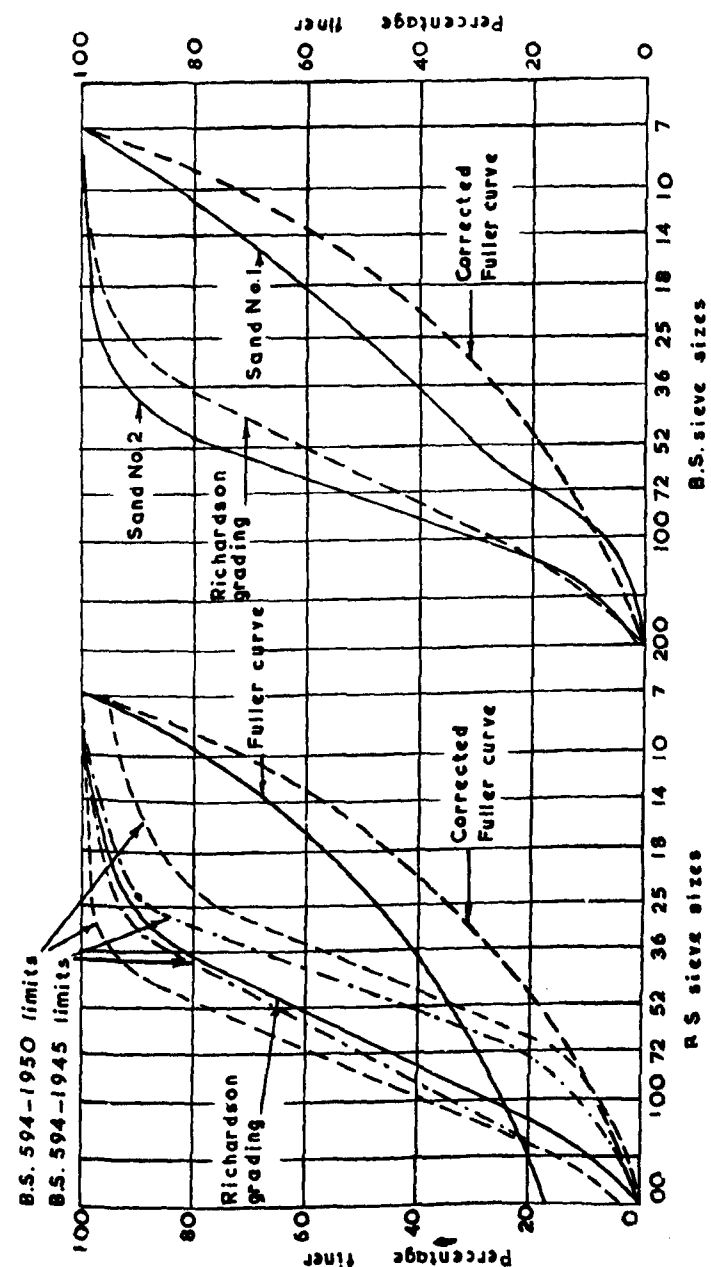


Fig. 5. Comparison of sand gradings

the grading limits for sand permitted by B. S. 594 (1950). The following facts can be observed:

(1) Sand No. 2 lies very close to the Clifford Richardson grading.

(2) The B.S. limits are based on the Clifford Richardson grading although sands coarser than this grading are permitted.

(3) The Fuller curve for sands passing No. 7 mesh shows that a much coarser sand than sand No. 2 would give a more compact mixture.

(4) Sand No. 1 is finer than the grading indicated by the Fuller curve except in material passing 100 mesh. 52 per cent passes the 25 mesh sieve as compared with 38 per cent indicated by the Fuller curve.

(5) The addition of Sand No. 2 to Sand No. 1 would not improve its grading.

6.3. A decision has now to be made on whether to follow Clifford Richardson and the British Standard or the more logical Fuller curve. As evidence that opinions differ on this matter in the United States, in Fig. 6 grading curves are plotted showing the specified gradings for sheet asphalt in four of the States. These gradings include the filler, or material passing the 200 mesh sieve. It will be seen that the general tendency is to specify a coarser sand than that allowed by the British Standard except in the case of Michigan. In the circumstances the best course is to prepare mixtures of the two sands available and determine the characteristics of the combinations.

6.4. To determine the voids in the sands and their combinations, mixtures were prepared using 8 parts by weight of 60/70 penetration bitumen to 100 parts of the sands. The following combinations were used:

Sample No.	Sand No. 1	Sand No. 2
A-1	100	Nil
A-2	80	20
A-3	60	40
A-4	40	60
A-5	20	80
A-6	Nil	100

Samples composed of 1000 grm. of dry sand and 80 grm. of bitumen were compacted at 300°F. using the standard Proctor compacting apparatus, 25 blows being given on each side of the 4½ in. diameter sample.

TABLE I

Asphalt Paving Mixture Design Experimental Series A

		1	2	3	4	5	6
1 Sand No. 1		1000	800	600	400	200	—
2 Sand No. 2		—	200	400	600	800	1000
3 Filler		80	80	80	80	80	80
4 Bitumen		1038	1056	1069	1074	1062	1065
5 Sample weight grm.		482	469	446	426	383	363
6 Weight in water		556	587	623	648	679	702
7 Volume - Cms ³	5-6 5	1.87	1.80	1.72	1.66	1.57	1.52
8 Specific gravity	7 7						
9 100 gr. mix volume	7 5 × 100	53.6	55.6	58.3	60.3	64	66
10 100 gr. materials vol.		42.1	42.1	42.1	42.1	42.1	42.1
11 Air volume	9-10	11.5	13.5	16.2	18.2	21.9	23.9
12 Percentage air voids	11 9 × 100	21.5	24.3	27.8	30.2	34.2	36.2
13 Percentage weight of bitumen		7.4	7.4	7.4	7.4	7.4	7.4
14 Percentage volume of bitumen	13 9 × 1.03	13.4	12.8	12.2	11.8	11.1	10.8
15 Percentage voids in aggregate	12+14	34.9	37.1	40.0	42.0	45.3	47.0
16 Soaked sample wt.		1193	1211	1222	1252	1258	1263
17 Weight in water		579	578	562	575	561	552
18 Absorbed water=A	16-5	155	155	153	178	196	198
19 Per cent absorbed water	18 5 × 100	14.9	14.7	14.4	16.6	18.5	18.6
20 Loss of weight=vol.	16-17	614	633	660	677	697	711
21 100 gr. saturated vol.	20 5 × 100	59.1	60.0	61.9	63.0	65.7	66.8
22 100 gr. materials vol.	=10	42.1	42.1	42.1	42.1	42.1	42.1
23 Air volume	21-22	17.0	17.9	19.8	20.9	23.6	24.7
24 Percentage air voids	23 21 × 100	28.8	29.8	32.0	33.2	36.0	36.9
25 Percentage volume of water	18 20 × 100	25.2	24.5	23.2	26.3	28.2	27.8
26 Swelling=S	20-7	58	46	37	29	18	11
27 Percentage swelling	20-7 7 × 100	10.4	7.9	5.9	4.5	2.4	1.6
28 Air displaced	18-26	97	109	116	149	180	187
29 Percentage water-filled air voids	28 7 × 100	17.4	18.6	18.6	23.0	26.5	26.6
30 Stability							
31 Resistance							

6.5. In Table 1 are shown the measurements and calculations made to arrive at the voids in the sand mixture. The opportunity was taken of measuring the water absorption of the mixtures and the amount of swelling that took place. The results, which are plotted in Fig. 7, show that any addition of Sand No. 2 to Sand No. 1 will result in an increase in voids and that more bitumen will be required to make a dense mixture with Sand No. 2 than with Sand No. 1. The results do however show that much more swelling takes place with Sand No. 1 than with Sand No. 2 and the amount varies with the proportions in which the sands are mixed. In other respects Sand No. 1 appears to be satisfactory and the mixture design will be developed assuming the use of this sand, care being taken that the swelling is kept within reasonable limits.

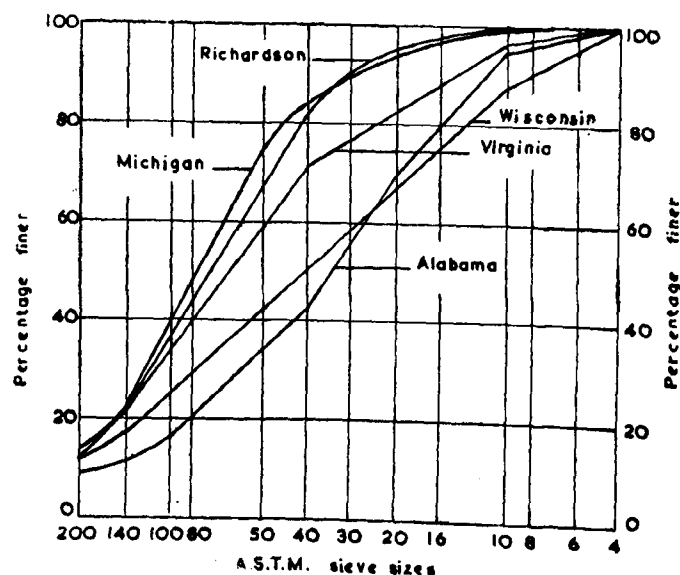


Fig. 6. American sand-filler gradings

6.6. Filler

There is a choice of two fillers, a limestone dust and a basalt stone dust. Their gradings and specific gravities are shown below:

	Limestone	Basalt
Total passing No. 72 mesh	100	100
" " No. 200 "	96.4	78.3
Specific gravity	2.70	2.92

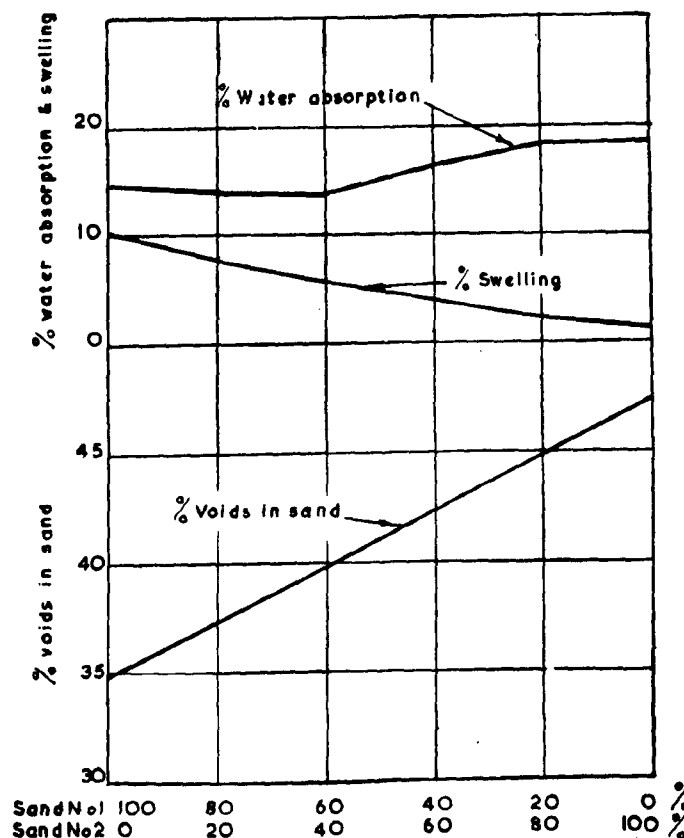


Fig. 7. Voids in sand mixtures

A decision has to be made on which filler to use and in what proportion. Reference has already been made to the difficulty of measuring voids but an indication of the effect of adding increasing quantities of filler to sand can be obtained by measuring compacted samples of sand, filler bitumen mixtures.

6.7. In Table 2 are shown the results of void tests made on mixtures of Sand No. 1 and limestone filler. The initial addition of filler appears to be the most effective in filling voids but it must be remembered that only from fully lubricated and thoroughly compacted mixture can the full effect of filler on voids be judged.

TABLE 2

**Asphalt Paving Mixture Design — Experimental
Series B.**

	1	2	3	4	5
1. Sand No. 1	950	900	850	800	750
2. Filler limestone	50	100	150	200	250
3. Bitumen 60/70 penetra- tion	90	90	90	90	90
4. Sample weight grm.	1072	1069	1058	1064	1060
5. Volume cms.	539	531	518	515	504
6. Vol. of 109 gr. mixture	54.80	54.2	53.5	52.8	51.8
7. Vol. of 100 gr. sand and filler	37.45	37.42	37.4	37.37	37.35
8. Voids in sand and filler	17.35	16.78	16.1	15.43	14.45
9. Percentage voids in sand and filler	31.6	30.9	30.1	29.2	27.9

**7. EXPERIMENTAL SERIES OF MIXTURES (SHEET
ASPHALT)**

7.1. Series C

In Table 3 and Fig. 8 are shown the results obtained from a series of mixtures prepared with Sand No. 1, 10 per cent of limestone filler and increasing proportions of 60/70 penetration bitumen. The first two increases in bitumen have little effect on the voids in the aggregate. Sample C 4 shows a marked reduction in aggregate voids and in air voids. This is reflected in the increase in specific gravity and the reduction in water absorption and swelling. In C 5 the air voids have been reduced to 5 per cent. The water absorption and swelling are very high in the leaner mixes and remain excessive even with the richest mix. The stability and the resistance to deformation show little variation through the series and remain low. These results appear to show that, with the method of consolidation used, no mixture in the series is satisfactory.

TABLE 3

**Asphalt Paving Mixture Design
Experimental Series C**

		1	2	3	4	5
1 Stone	s. g.					
2 Sand No. 1	2.67	900	900	900	900	900
3 Filler Limestone	2.70	100	100	100	100	100
4 Bitumen 60/70 pen.	1.03	90	100	110	120	130
5 Sample Weight grm		1069	1071	1081	1093	1102
6 Weight in water		538	536	540	578	586
7 Volume — Cms ³	5-6	531	535	541	515	516
8 Specific gravity	$\frac{5}{7}$	2.01	2.01	2.06	2.12	2.13
9 100 gr. mix volume	$\frac{7}{5} \times 100$	49.8	49.8	48.6	47.2	47.0
10 100 gr. materials volume		42.25	42.8	43.4	43.8	44.25
11 Air volume	9-10	7.55	7.0	5.2	3.4	2.25
12 Percentage air voids	$\frac{11}{9} \times 100$	16.1	14.0	10.7	7.2	4.8
13 Percentage weight of bitumen		8.25	9.1	9.9	10.7	11.5
14 Percentage volume of bitumen	$\frac{13}{9} \times \frac{100}{1.03}$	16.1	17.7	19.8	22.0	23.7
15 Percentage voids in ag- gregate	12 + 14	31.2	31.7	30.5	29.2	28.5
16 Soaked sample weight		1180	1183	1183	1147	1146
17 Weight in water		591	594	598	609	618
18 Absorbed water=A	16-5	111	112	102	54	44
19 Percentage absorbed water	$\frac{18}{5} \times 100$	10.4	10.45	9.4	4.9	4.0
20 Loss of weight=vol.	16-17	589	589	585	538	528
21 100 gr. saturated vol.	$\frac{20}{5} \times 100$	55.1	55.0	54.0	49.2	48.0
22 100 gr. materials vol.	=10	42.25	42.8	43.4	43.8	44.25
23 Air volume	21-22	12.85	12.2	10.6	5.4	3.75
24 Percentage air voids	$\frac{23}{21} \times 100$	23.2	22.2	19.6	11.0	7.9
25 Percentage volume of water	$\frac{18}{20} \times 100$	18.9	19.0	17.4	9.25	8.3
26 Swelling=S	20-7	58	54	44	23	12
27 Percentage swelling	$\frac{20-7}{7} \times 100$	10.9	10.1	8.1	4.5	2.3
28 Air displaced	18-26	53	58	58	31	32
29 Percentage waterfilled air voids	$\frac{28}{7} \times 100$	10.0	10.8	10.7	6.0	6.2
30 Stability		435	295	295	295	335
31 Resistance		38.75	26.25	26.25	26.25	37.50

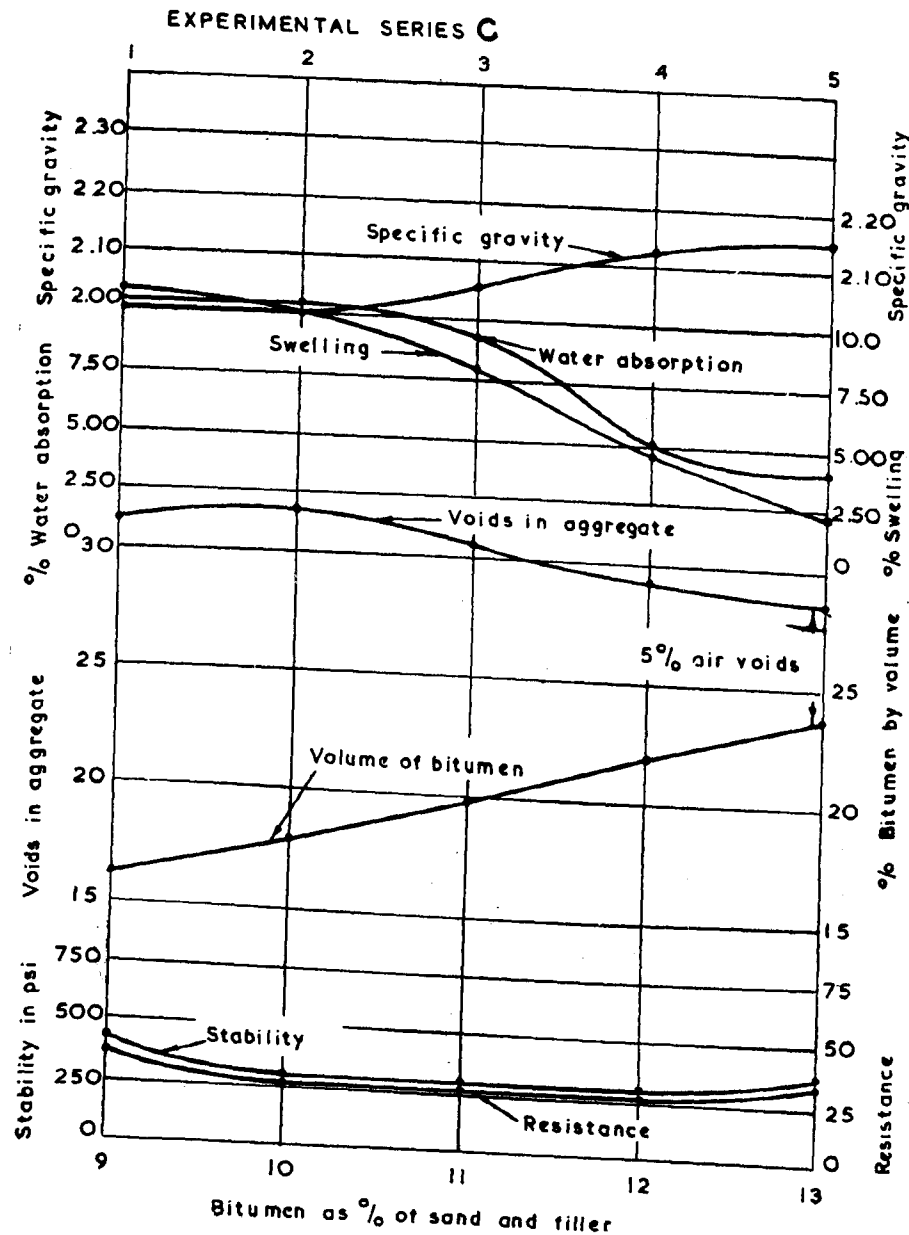


Fig. 8.

7.2. Series D

It has been described earlier how Macnaughton reduces the bitumen requirement in his sheet asphalt mixture by increasing the filler. The effect of such an increase is shown in Table 4 & Fig. 9 which show the test results obtained with Series D. The filler has been increased to 20 per cent of the weight of aggregate with marked beneficial results. From a diagram such as this, an engineer can choose a mixture suited to the traffic and weather conditions to which the surfacing will be subjected. The decision which must usually be made is on the extent to which stability must be sacrificed to density. The mixture chosen would probably be D 3 with 3.56 per cent of air voids and a stability of 710, a figure very little below the maximum. The bitumen content on the total mix is 9.9 per cent. If the traffic was very heavy and an appreciable amount of further consolidation could be expected on the road, then a mixture half way between D 2 and D 3 could be chosen. This mixture would contain 9.5 per cent of bitumen and would have 5 per cent of air voids. It should also be noted for comparison with other mixtures that D 3 will absorb 1.2 per cent of water and will swell by 0.2 per cent.

7.3. Fig. 10 shows the results of the stability tests on these mixtures. These curves show very clearly that deformation commences at very low pressures. Usually upto 0.01 inch is elastic. It will be noted that four of the curves show a change in slope and in computing the resistance the higher value has been taken.

7.4. As there is a choice of two fillers, Series E was prepared in which basalt stone dust filler replaced the limestone filler used in Series D. The results are shown in Fig. 11. Only in one respect are the results similar to those obtained in Series D. The air voids are reduced just below 5 per cent in sample E 3 but from this aspect only could mixture E 3 be considered as satisfactory. The water absorption and swelling are still very high with this sample, being 4.5 per cent and 4.90 per cent respectively. The stability is low, being 400. One mixture only might be considered; E 4 has a reasonable water absorption but the voids are so low that the mix might become unstable if the mixture is further consolidated under heavy traffic. The bitumen content of mixture E 4 is 10.7 per cent as compared with 9.9 per cent for a very satisfactory mixture containing limestone dust filler. Taken by itself, the greater bitumen requirement of stone dust filler would not eliminate this material as a low filler cost might offset the extra cost of bitumen. The real objection is to the high water absorption and swelling which accompany any slight reduction in bitumen content, reductions which might inadvertently occur in full scale operations.

TABLE 4

Asphalt Paving Mixture Design
Experimental Series D

		1	2	3	4	5
1 Stone	s. g.					
2 Sand No. 1	2.67	800	800	800	800	800
3 Filler Limestone	2.70	200	200	200	200	200
4 Bitumen 60/70 pen.	1.03	90	100	110	120	130
5 Sample weight gm		1064	1070	1079	1082	1081
6 Weight in water		548	580	595	597	597
7 Volume - Cms ³	5-6	516	490	484	485	484
8 Specific gravity	$\frac{7}{5}$	2.06	2.18	2.22	2.22	2.22
9 100 gr. mix volume	$\frac{7}{5} \times 100$	48.5	45.8	45.0	45.0	45.0
10 100 gr. materials volume		42.25	42.8	43.4	43.8	44.25
11 Air volume	9-10	6.25	3.0	1.6	1.2	0.75
12 Per cent air voids	$\frac{11}{9} \times 100$	12.9	6.55	3.56	2.67	1.67
13 Per cent weight of bitumen		8.25	9.1	9.9	10.7	11.5
14 Per cent volume of bitumen	$\frac{13}{9} \times \frac{100}{1.03}$	16.5	19.3	21.4	23.1	24.8
15 Per cent voids in aggregate	12+14	29.4	25.85	24.96	25.77	26.47
16 Soaked sample weight		1146	1103	1092	1091	1085
17 Weight in water		600	608	607	605	600
18 Absorbed water=A	16-5	82	33	13	9	4
19 Per cent absorbed water	$\frac{18}{5} \times 100$	7.7	3.1	1.2	0.83	0.37
20 Loss of weight=vol.	16-17	546	495	485	486	485
21 100 gr. saturated vol.	$\frac{20}{5} \times 100$	51.4	46.2	45.0	45.0	45.0
22 100 gr. materials vol.	=10	42.25	42.8	43.4	43.8	44.25
23 Air volume	21-22	9.15	3.4	1.6	1.2	0.75
24 Per cent air voids	$\frac{23}{21} \times 100$	17.8	7.37	3.56	2.67	1.67
25 Per cent volume of water	$\frac{18}{20} \times 100$	15.0	6.67	2.68	1.85	0.83
26 Swelling=S	20-7	30	5	1	1	1
27 Per cent swelling	$\frac{20-7}{7} \times 100$	5.8	1.02	0.2	0.2	0.2
28 Air displaced	18-26	52	28	12	8	3
29 Per cent waterfilled air voids	$\frac{28}{7} \times 100$	10.1	5.7	2.5	1.65	0.62
30 Stability		530	765	710	490	420
31 Resistance		57.5	85	60	32.5	25

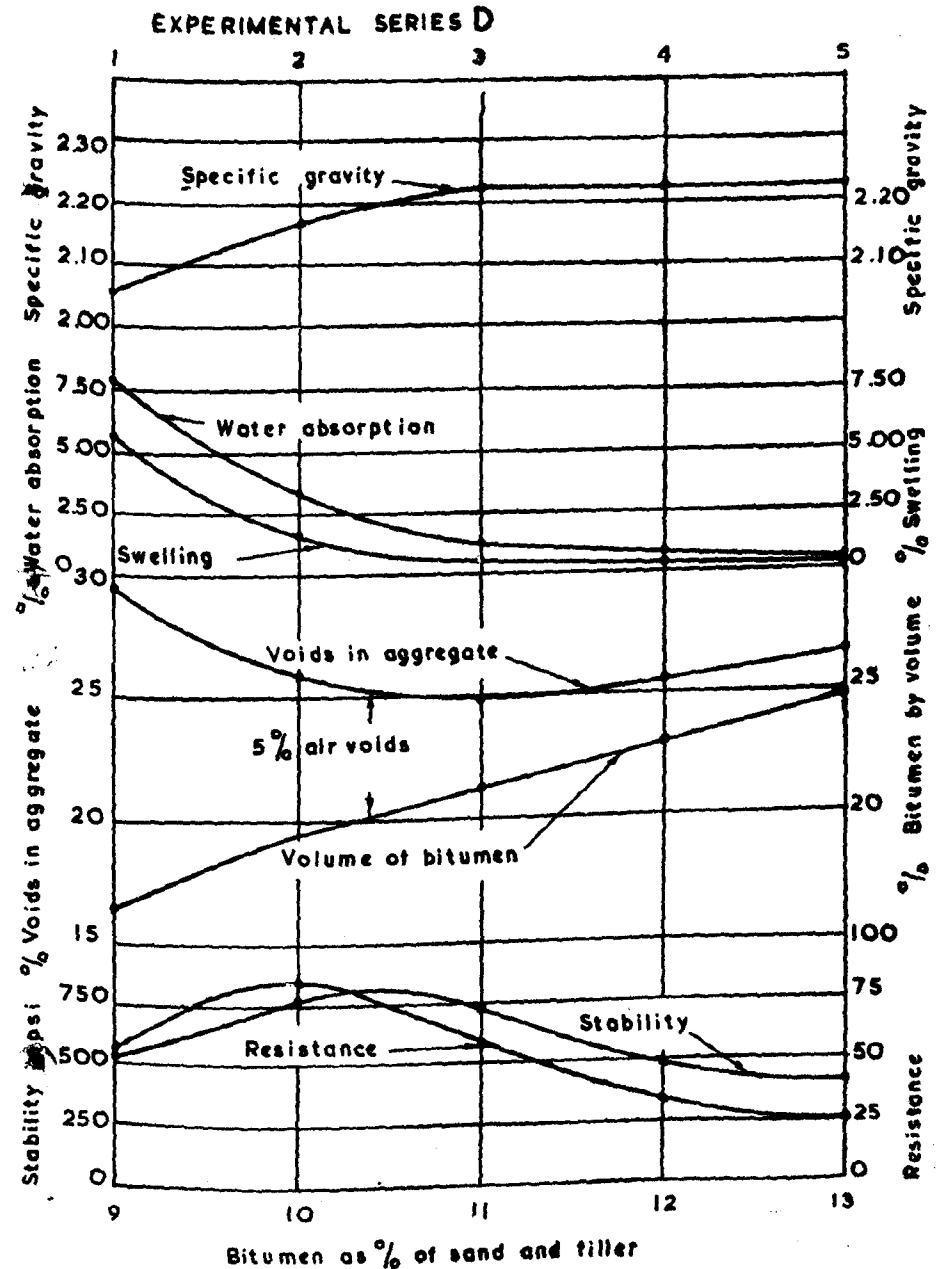
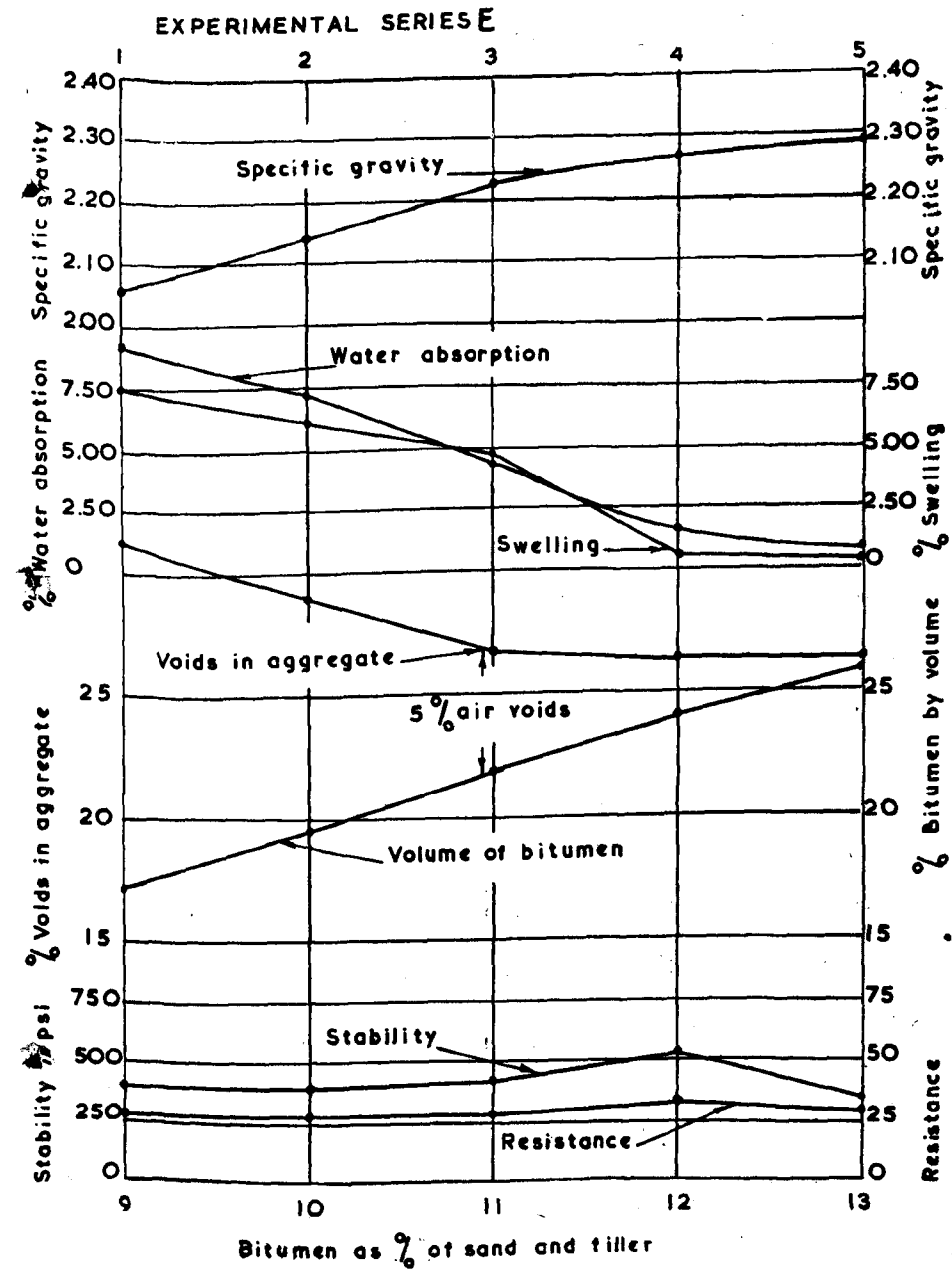
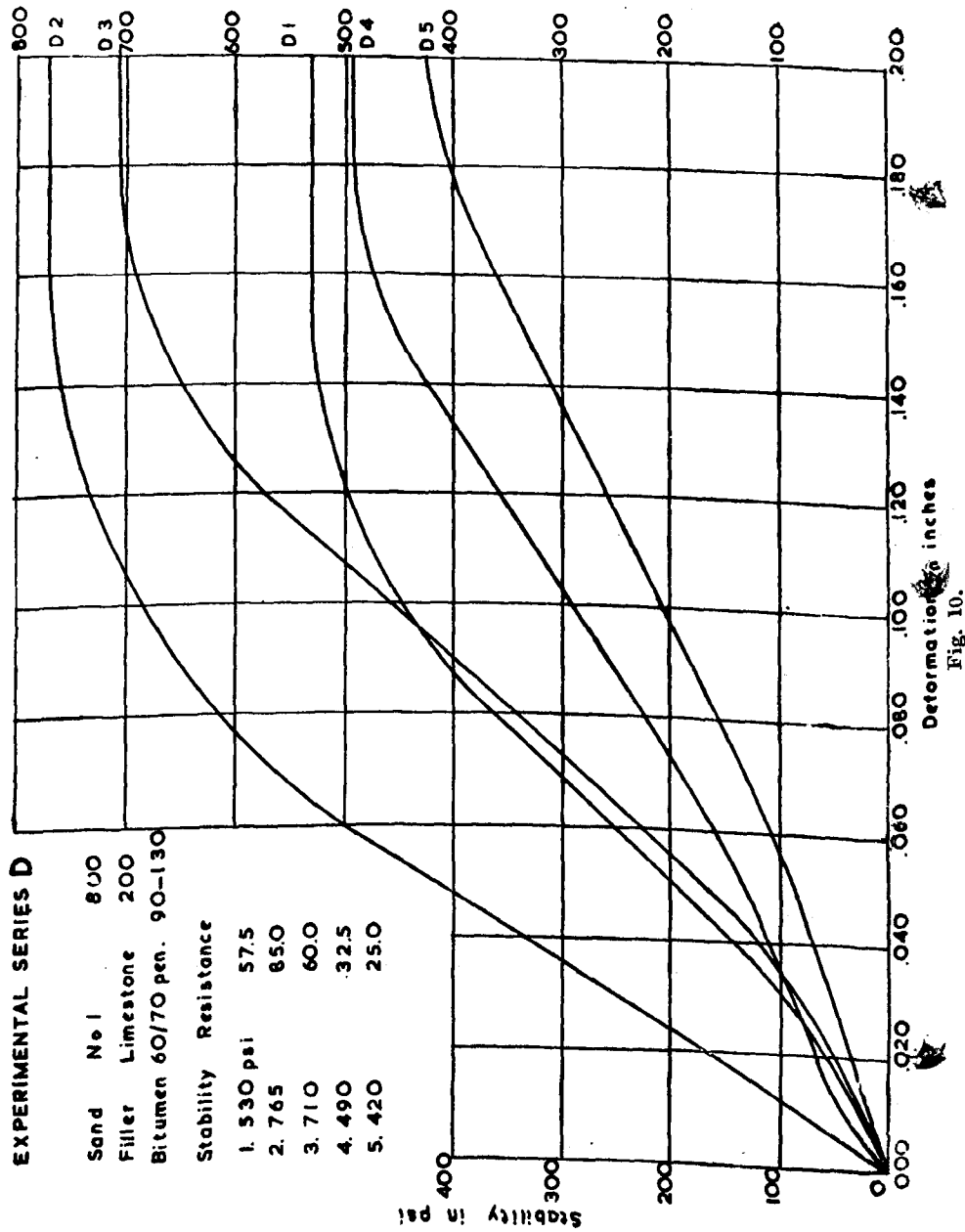


Fig. 9.



7.5. Series F

Having decided on the sand and the filler to be used in the sand sheet asphalt mixture, we have to confirm that the bitumen penetration of 60/70 is satisfactory. Series F mixtures are exactly the same as Series D except that 30/40 penetration bitumen has been used. The test results are given in Table 5 and are plotted in Fig. 12. The stability test curves are shown in Fig. 13. If Fig. 12 is compared with Fig. 9 it will be seen that there are few pronounced differences between the two diagrams. The exceptions are that swelling and water absorption in the leaner mixtures are reduced when the 30/40 grade of bitumen is used and that the stability and resistance to deformation are better maintained in the richer mixtures of F series.

7.6. A comparison of Figs. 10 and 13 reveals that there is considerably less variation of stability values with bitumen content in the case of F series and that mixtures F 3, F 4 and F 5 were not broken during the stability test. Although mixture D 3 gave a higher stability than F 3, the F 3 sample remained unbroken after the test. Before a final decision is made, it will be advisable to check series F mixtures when subjected to a greater compactive effort.

7.7. Series F. B.

The mixtures in Series F.B are identical with those in Series F but instead of 25 blows of the Proctor tamper, 50 blows were given to each side of the sample. The test results are shown in Fig. 14. The effects of the increased compactive effort are (a) a reduction of swelling and a pronounced increase in stability and in resistance to deformation in the leaner mixtures (b) a reduction in voids in the aggregate. It will be seen that the increased compactive effort has little effect on the characteristics of the mixtures at the rich end of the range.

7.8. Composition of Sand Sheet Asphalt using Materials available

A sand sheet asphalt for general use prepared with the materials available should have the following composition :

		Per cent
Sand No. 1	80 parts	72.1
Limestone filler	20 „	18.0
30/40 penetration bitumen	11 „	9.9
	<u>111</u>	<u>100.0</u>

TABLE 5

Asphalt Paving Mixture Design
Experimental Series F

		1	2	3	4	5
1	Stone	s. g.				
2	Sand No. 1	2.67	800	800	800	800
3	Filler Limestone	2.70	200	200	200	200
4	Bitumen 30/40 pen.	1.03	90	100	110	120
5	Sample weight grm		1061	1067	1089	1088
6	Weight in water		566	582	599	603
7	Volume-Cms ³	5-6	495	485	490	485
8	Specific gravity	5				
		7	2.15	2.20	2.22	2.24
9	100 gr. mix volume	7				
		5	46.5	45.5	45.0	44.7
10	100 gr. materials vol.		42.25	42.8	43.4	43.8
11	Air volume	9-10	4.25	2.7	1.6	0.9
12	Percentage air voids	11				
		9	9.15	5.95	3.55	2.02
13	Percentage weight of bitumen		8.25	9.1	9.9	10.7
14	Percentage volume of bitumen	13				
		9	17.25	19.4	21.4	23.3
15	Percentage voids in aggregate	12 + 14	26.40	25.35	24.95	25.32
16	Soaked Sample weight		1104	1090	1100	1094
17	Weight in water		601	600	603	606
18	Absorbed water = A	16-5	43	23	11	6
19	Percentage absorbed water	18				
		5	4.05	2.16	1.0	0.55
20	Loss of weight = vol.	16-17	503	490	497	488
21	100 gr. saturated vol.	20				
		5	47.5	46.0	45.6	45.0
22	100 gr. materials vol.	= 10	42.25	42.8	43.4	43.8
23	Air volume	21-22	5.25	3.2	2.2	1.2
24	Percentage air voids	23				
		21	11.0	6.95	4.8	2.7
25	Percentage volume of water	18				
		20	8.6	4.7	2.2	1.25
26	Swelling=S	20-7	8	5	7	3
27	Percentage swelling	20-7				
		7	1.6	1.03	1.4	0.6
28	Air displaced	18-26	35	18	4	3
29	Percentage waterfilled air voids	28				
		7	7.1	3.71	0.84	0.61
30	Stability		640	840	630	585
31	Resistance		57.5	75	40	40

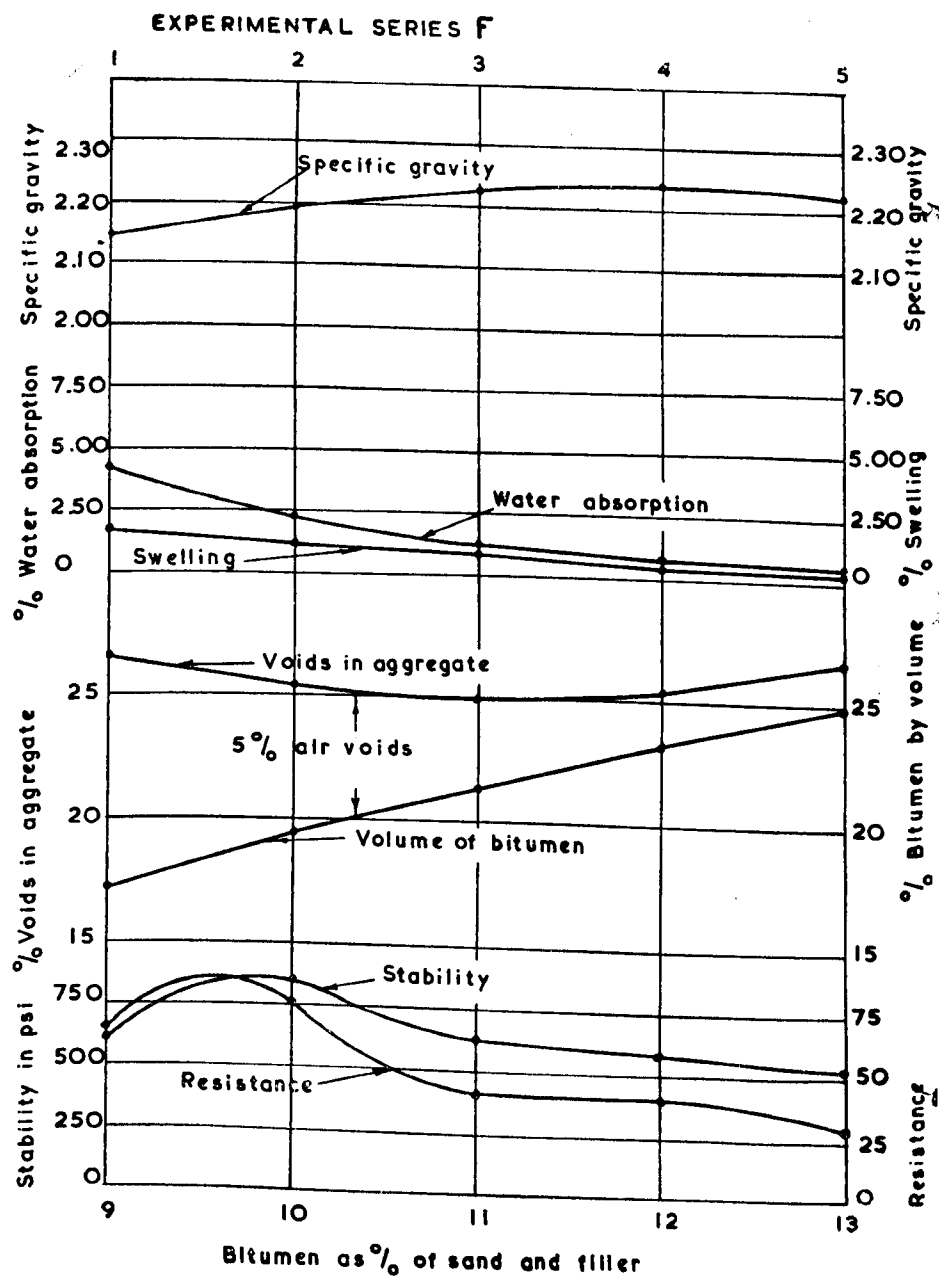
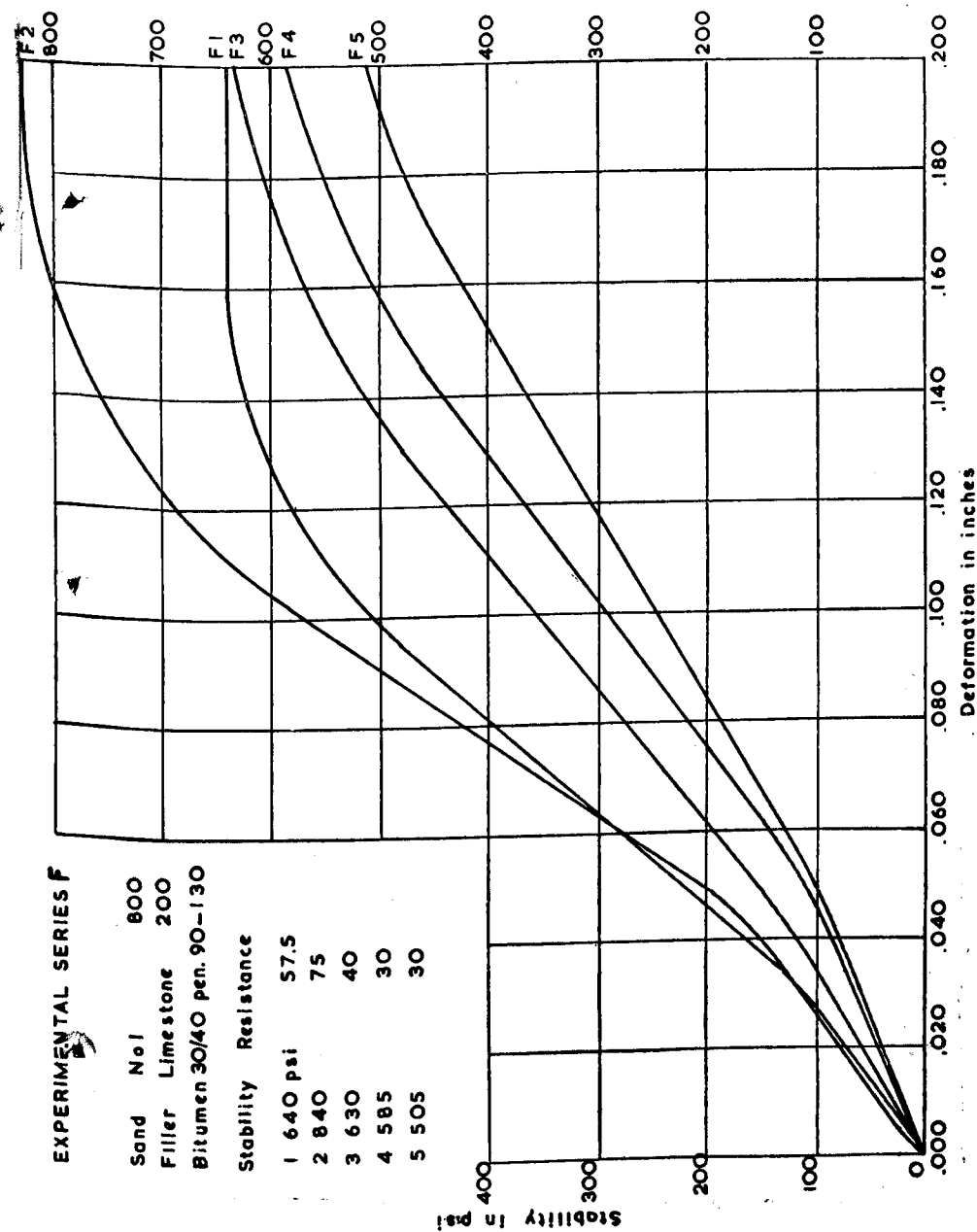


Fig. 12.



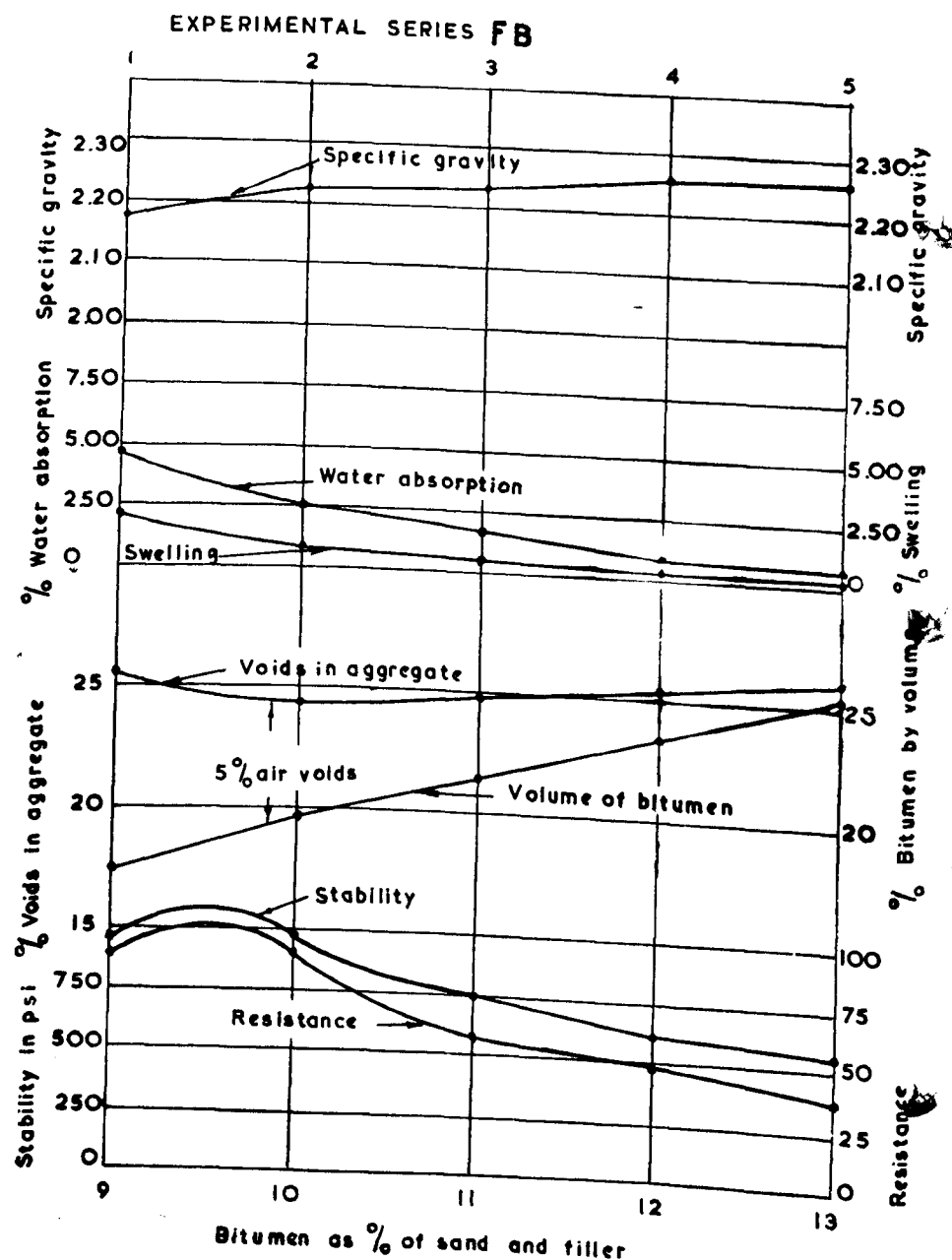


Fig. 14.

At bus stops or places where high stability is required, the bitumen could be reduced so that the mixture would correspond to one half way between F 2 and F 3. The composition would then be :

		Per cent
Sand No. 1	80 parts	72.4
Limestone filler	20 „	18.1
30/40 penetration bitumen	10.5 „	9.5
	110.5	100.0

8. EXPERIMENTAL SERIES OF MIXTURES (STONE FILLED SHEET ASPHALT)

8.1. Series H.

Although the mixtures already designed are very satisfactory as engineering materials, an engineer must be assured before employing them that they are also economical and could not be effectively replaced by cheaper mixtures. Reference has been made to stone-filled asphalts in which the stone forms no part of the internal structure of the mix and acts as a displacer. Series H is exactly the same as Series F, except that 250 grms. of $\frac{3}{4}$ in. stone have been added to the 800 grms. of Sand No. 1 and 200 grms. of limestone filler. The bitumen remains the same as in Series F. The test results are given in Table 6 and are plotted in Fig. 15. The diagram shows very clearly that these mixtures are difficult to compact. The voids are extremely high and in consequence the water absorption and swelling are high and the stability is low. Adequate compaction may be obtained by increasing the compactive effort or reducing the viscosity of the binder. The marked effect of increasing the compactive effort in the case of Series F has already been seen.

8.2. Series H.B.

In Series H. B the compactive effort has been increased by applying 50 blows of the tamper on each side of the sample. The test results are shown in Fig. 16. The diagram is now of normal form and shows a pronounced reduction in voids in the aggregate following the introduction of the stone. The most interesting change is the increase in stability and resistance to deformation. Whilst the stability results are almost identical with those of Series F. B., the resistance to deformation reaches very high values. Mixture H. B. is therefore satisfactory if properly compacted and further tests showed that compaction was

TABLE 6

Asphalt Paving Mixture Design Experimental Series H

		1	2	3	4	5
1 Stone passing 3/4 in.	s. g.					
2 Sand No. 1	2.88	250	250	250	250	250
3 Filler Limestone	2.67	800	800	800	800	800
4 Bitumen 30/40 pen.	2.70	200	200	200	200	200
5 Sample weight grm.	1.03	90	100	110	120	130
6 Weight in water		1011.3	1007	1006	999	1000
7 Volume—Cms ³		537.3	530	542	546	561
8 Specific gravity	5-6	474	477	464	453	439
	7	2.13	2.11	2.18	2.20	2.28
9 100 gr. mix volume	$\frac{7}{5} \times 100$	46.9	47.4	46.2	45.3	43.8
10 100 gr. materials volume		40.9	41.3	41.7	42.1	42.5
11 Air volume	9-10	6.0	6.1	4.5	3.2	1.3
12 Per cent air voids	$\frac{11}{9} \times 100$	12.8	12.9	9.75	6.85	2.96
13 Per cent weight of bitumen		6.7	7.4	8.1	8.75	9.4
14 Per cent volume of bitumen	$\frac{13}{9} \times 100$	13.9	15.2	17.0	18.75	20.85
15 Per cent voids in aggregate	$\frac{12+14}{1.03}$	26.7	28.1	26.75	25.60	22.91
16 Soaked sample weight		1069.8	1064.4	1046.4	1032	1015.6
17 Weight in water		585	574	579.7	579	575.2
18 Absorbed water=A	16-5	58.5	57.4	40.4	33	15.6
19 Per cent absorbed water	$\frac{18}{5} \times 100$	5.8	5.7	4.0	3.3	1.56
20 Loss of weight=volume	16-17	484.8	490.4	466.7	453	440.4
21 100 gr. saturated volume	$\frac{20}{5} \times 100$	48	48.7	46.5	45.4	44.0
22 100 gr. materials volume	=10	40.9	41.3	41.7	42.1	42.5
23 Air volume	21-22	7.1	7.4	4.8	3.3	1.5
24 Per cent air voids	$\frac{23}{21} \times 100$	14.8	15.2	10.6	7.3	3.4
25 Per cent volume of water	$\frac{18}{20} \times 100$	12.1	11.7	8.7	7.3	3.5
26 Swelling=S	20-7	10.8	13.4	2.7	—	1.4
27 Per cent swelling	$\frac{20-7}{7} \times 100$	2.3	2.8	0.6	—	0.3
28 Air displaced	18-26	47.7	44.0	37.7	33	14.2
29 Per cent waterfilled air voids	$\frac{28}{7} \times 100$	10.0	9.2	8.7	7.3	3.2
30 Stability		576	566	390	342	512
31 Resistance		65	65	30	30	45

EXPERIMENTAL SERIES H

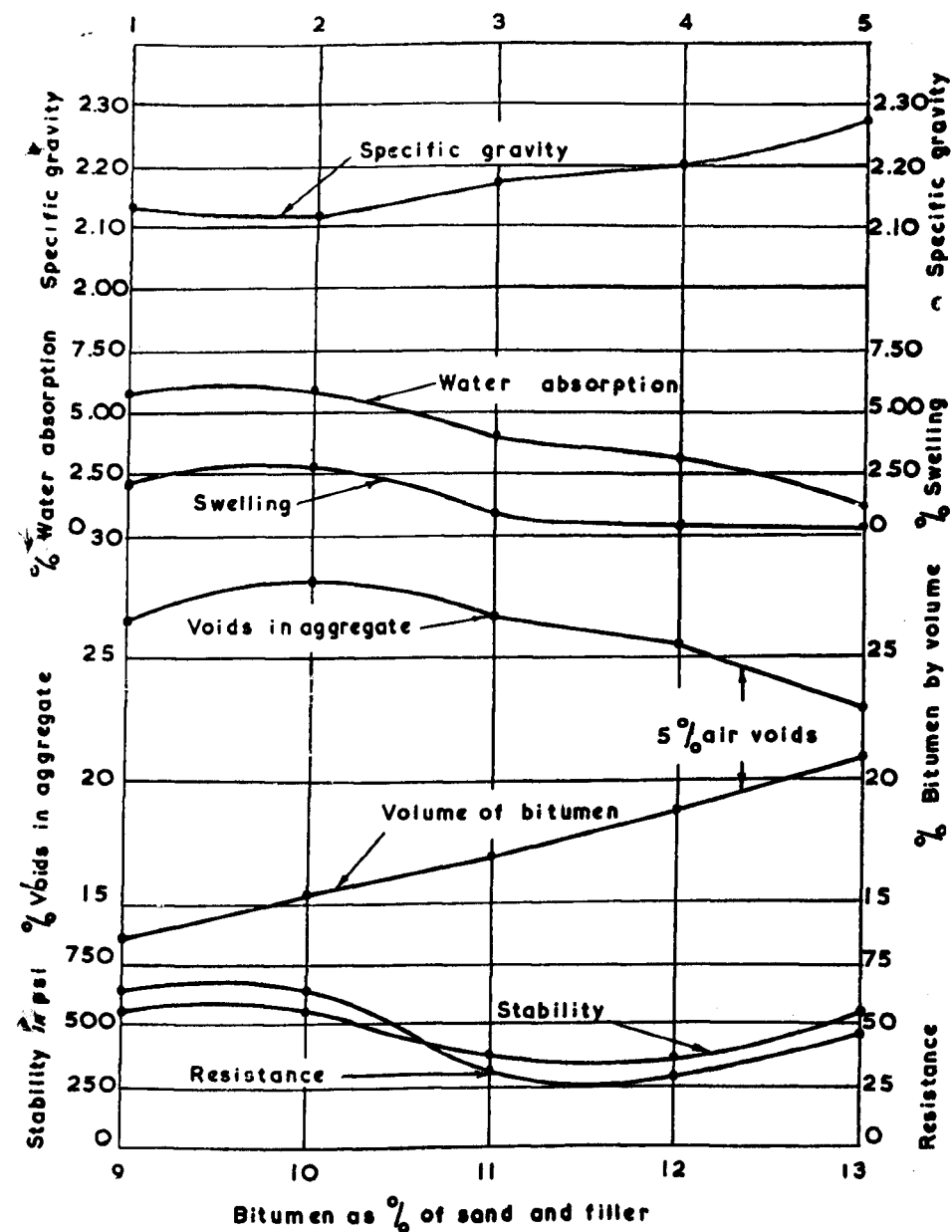


Fig. 15.

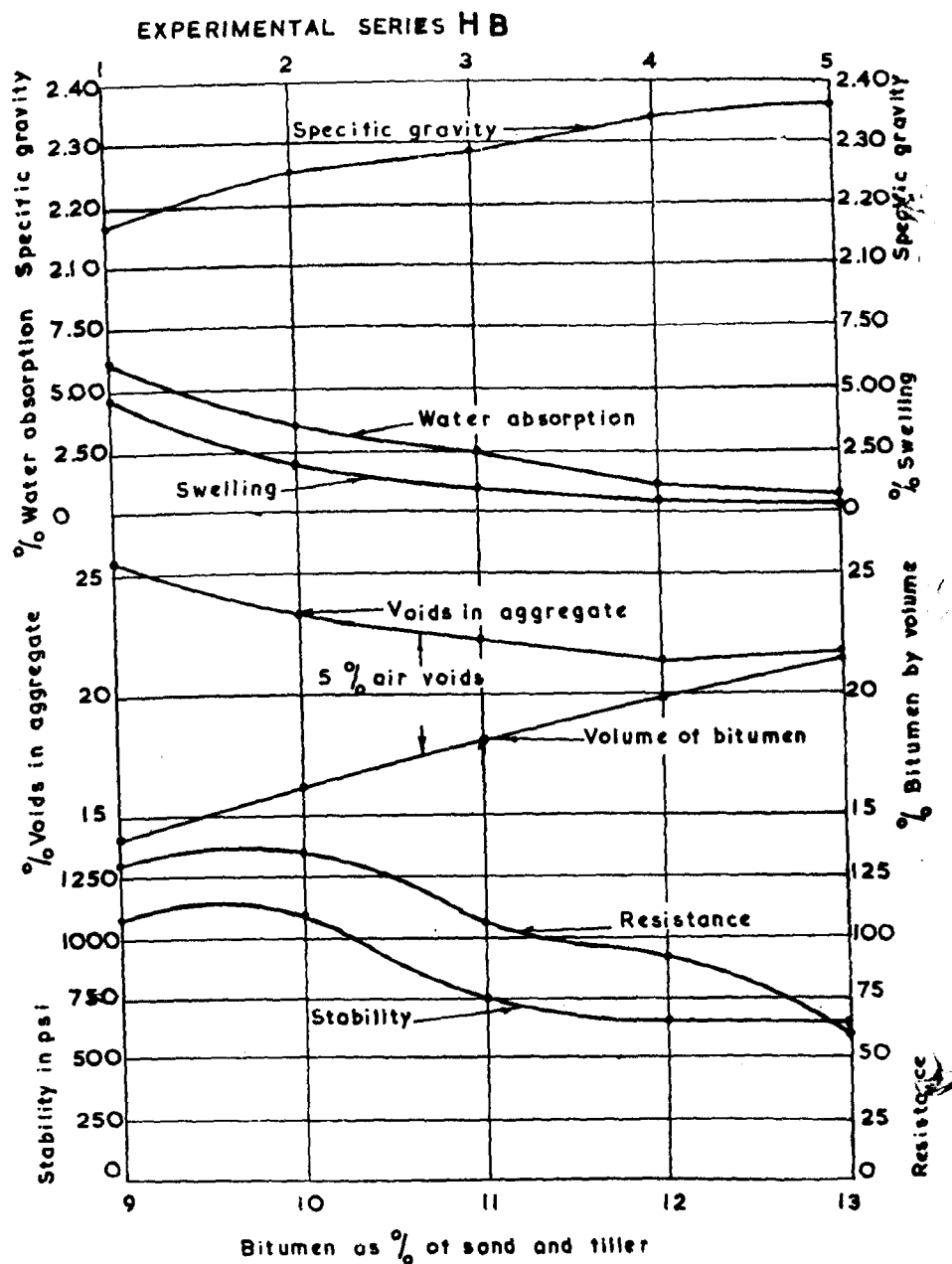


Fig. 16.

improved by substituting 60/70 penetration bitumen for the 30/40 bitumen used in Series H.

8.3. Composition of stone filled sheet asphalt

The composition of the stone filled sheet asphalt should therefore be the following:

	Per cent	Per cent
3/4 in. stone	25	18.4
Sand No. 1	80	58.8
Limestone filler	20	14.7
60/70 penetration bitumen	11	8.1
	<u>136</u>	<u>100.0</u>

9. ASPHALTIC CONCRETE

9.1. It has been shown that the optimum bitumen content of an asphalt mixture can be reduced by adding stone as a displacer in sand sheet asphalt. If the stone is graded and combined with sand and filler so that the aggregate has a grading approximating to the Fuller curve applicable to the maximum stone size used, the optimum bitumen content for that grading can become as low as 4.5 per cent by weight of the mixture. Mixtures of this type, known as asphaltic concrete, are being laid all over the world in very large quantities.

9.2. Aggregate Grading

There are many different opinions on the grading which should be adopted for the aggregate to be used in asphaltic concrete. In Fig. 17 are shown gradings specified by the States of Michigan and Alabama. A typical Fuller curve is shown for comparison. It will be seen, as was also noticed in the case of sand gradings for sheet asphalt, that the Alabama grading tends to follow the Fuller curve except that the proportion of sand to stone is greater than that indicated by the Fuller curve. The Michigan gradings are unusual in that aggregate passing 10 mesh and retained on 40 mesh (A. S. T. M. sieves) is omitted. It was shown in Fig. 6 that the sand used in Michigan for sheet asphalt approximated to the Richardson grading and therefore had only a small percentage retained on the 40 mesh sieve. The combination of this sand with crushed stone will result in a grading curve as shown for Michigan. The omission of material intermediate between sand and stone is thought by many engineers to render the mixture less harsh and to assist compaction.

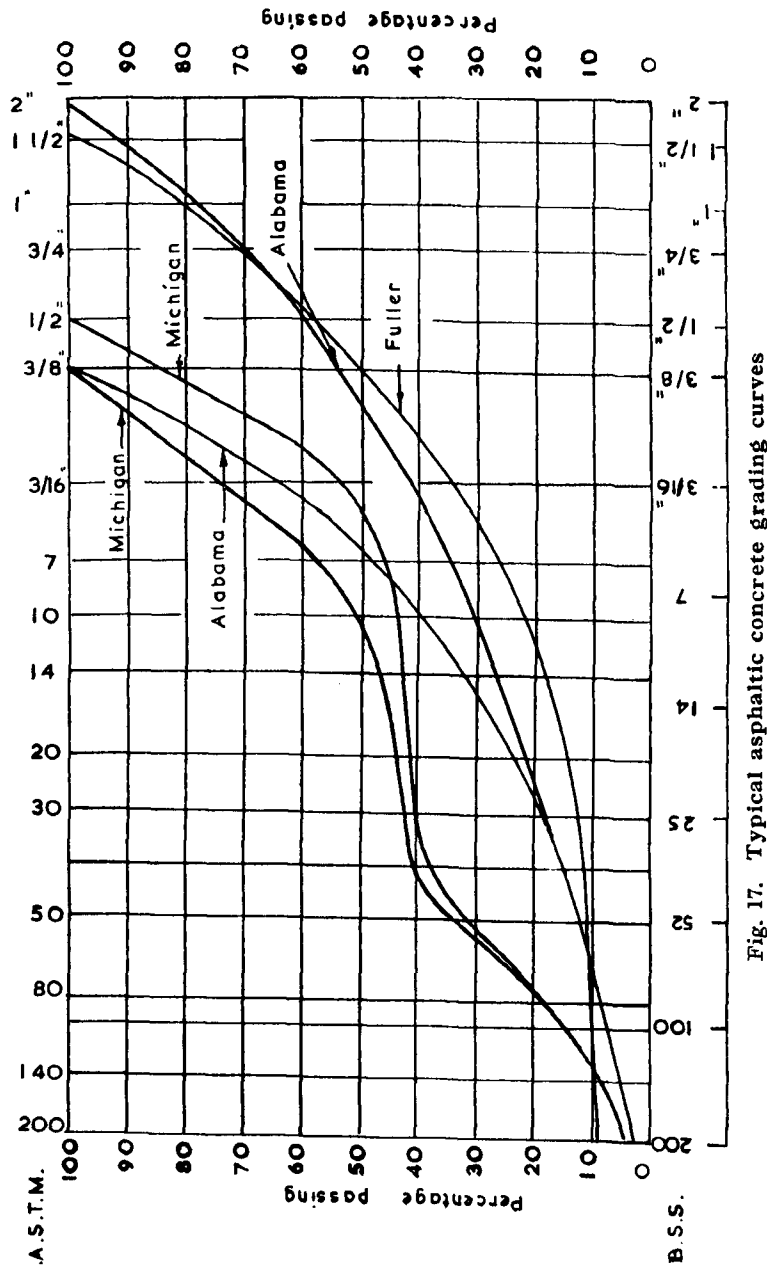


Fig. 17. Typical asphaltic concrete grading curves

9.3. Gradings for Experimental Work

The gradings of the sands available have already been given. The type of asphaltic concrete having the most general application is one laid from 1½ in. to 2 in. thick and having an aggregate passing ¾ in. mesh. A crusher run stone is available having the following grading:

	Per cent
Passing ¾ in.	100
„ 3/8 in.	61.7
„ 3/16 in.	31.5
„ No. 7 B. S. sieve	14.7
„ No. 14 „ „	8.3
„ No. 25 „ „	5.4
„ No. 52 „ „	4.1
„ No. 100 „ „	3.1
„ No. 200 „ „	2.2

9.4. A problem which immediately arises is how to determine the proportions in which available materials of known gradings should be mixed so that the combined grading approaches as nearly as possible to the desired grading. This is a problem also met in soil mechanics and in concrete designing and the method of Rothfuchs is frequently used for its solution. Full descriptions of this method are readily available¹ but, briefly, the method involves the use of a scale of sieve sizes such that the desired cumulative curve plots as a straight line. To this scale the cumulative gradings of the available materials are plotted and for the curves thus found are substituted straight lines such that the areas between the curve and the straight line are balanced. When the opposite ends of these straight lines are joined together, the proportions required of the available materials can be read off from the intersections of the joining lines with the straight line representing the desired grading.

9.5. Gradings for Binder Courses

It is generally agreed that an asphaltic concrete mixture which has a grading approaching closely to the Fuller curve is somewhat harsh and, although such mixtures are very dense, they may show, in view of the very low optimum bitumen content, a tendency to unravel under traffic. Such mixtures are nevertheless frequently used as binder courses under sheet asphalt and stone filled sheet asphalt or as a regulating course under asphaltic concrete. The method of Rothfuchs is used to determine the proportions of the available materials which would be required

to give an aggregate grading approximating to that indicated by the Fuller curve.

The best combinations are	Per cent	Per cent
3/4 in. stone	76	82
Sand No. 1	16	—
Sand No. 2	—	10
Limestone filler	8	8
	100	100

9.6. Gradings for Asphaltic Concrete Wearing Courses

For a city with a heavy rainfall, it is advisable to avoid in wearing course mixtures gradings which have very low optimum bitumen contents. A grading has recently been suggested for use in India⁹ and the possibility of adopting it will therefore be examined. The grading is as follows:

Passing	1 in.	Per cent	Mean value Per cent
		100	100
"	3/4 in.	90-100	95
"	1/2 in.	75-90	82.5
"	3/8 in.	62-83	72.5
"	3/16 in.	44-62	53
"	No. 7 B. S. sieve	35-50	40.25
"	No. 14 "	29-42	35.5
"	No. 25 "	24-36	30
"	No. 52 "	18-30	24
"	No. 100 "	12-24	18
"	No. 200 "	4-12	8

9.7. The grading curve is plotted in Fig. 18. The flattening of the curve between sieve sizes No. 52 and No. 7 indicates that a fine sand would be required if the mean values of the range are to be obtained. Sand No. 2 could therefore with advantage be used. Although the mean values of the sand range could not be obtained with Sand No. 1, this well graded sand could be used to give a mixture within the specified range and for the purpose of making a determination of the aggregate proportion by the Rothfuchs method, the points on the curve representing material passing the No. 14 and the No. 200 sieves may be joined by a

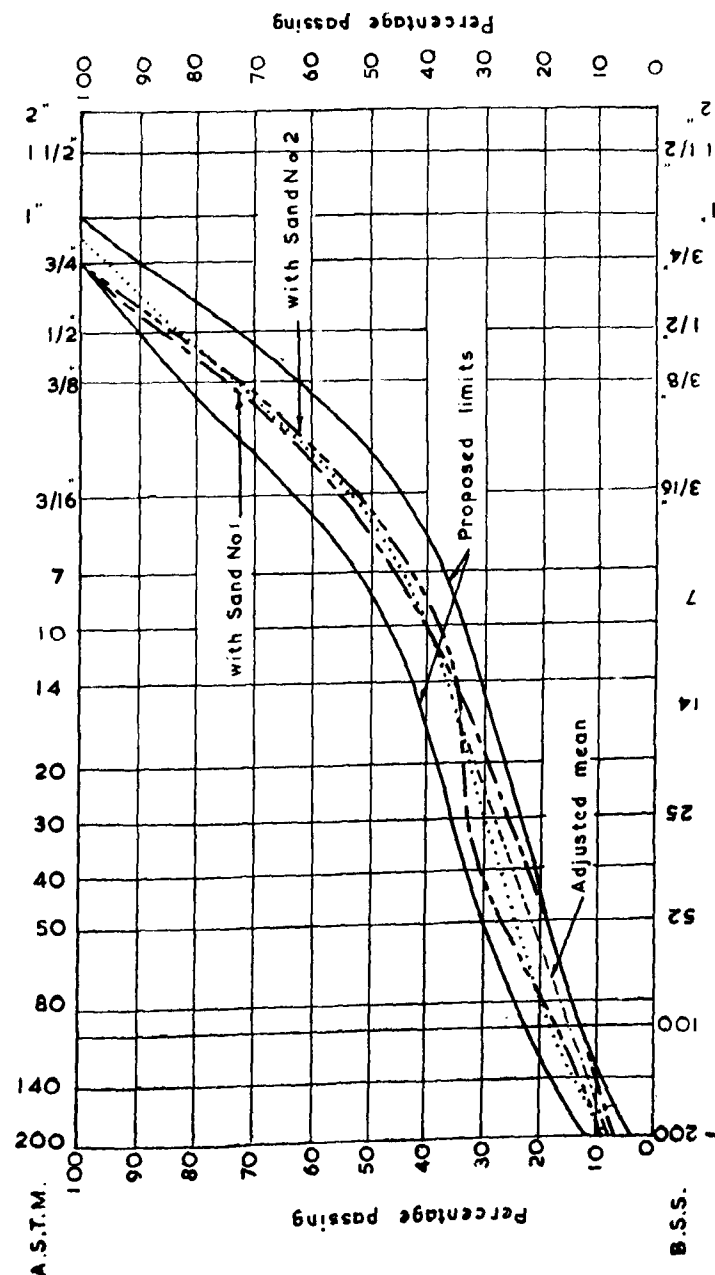


Fig. 18. Grading curve for city in India

straight line. The mean values for the sand filler range then become

Passing No.	7 B. S. seive	Per cent
		40.25
"	No. 14	35.5
"	No. 25	27.5
"	No. 52	21.0
"	No. 100	14.0
"	No. 200	8.0

9.8. Proportions of Available Aggregates

In Fig. 19 is shown the application of the Rothfuchs method to the materials available. The following proportions are indicated:

	Per cent	Per cent
3/4 in. crusher run stone	66	71
Sand No. 1	26	—
Sand No. 2	—	22
Limestone filler	8	7
	100	100

The combined aggregate gradings given by these proportions are shown in Table 7 and the grading curves are plotted in Fig. 18. Either of these gradings would satisfy the specification and it only remains to try experimental series of mixtures employing these aggregates.

10. ASPHALTIC CONCRETE EXPERIMENTAL MIXTURES

10.1. The materials used in the mixtures now to be considered were the stone, the two sands and limestone filler, mixed in the proportions already ascertained together with 60/70 penetration bitumen. The quantity of bitumen used was much less than that required in the sheet asphalt and stone filled series of mixtures. The series tested were the following.

Series	3/4 in. Stone	Sand No. 1	Sand No. 2	Limestone Filler	60/70 penc. bitumen
J	820 gm.		100 gm.	80 gm.	41-69 gm.
K	660 "	260 gm.		80 "	48-76 "
L	710 "		220 "	70 "	48-76 "

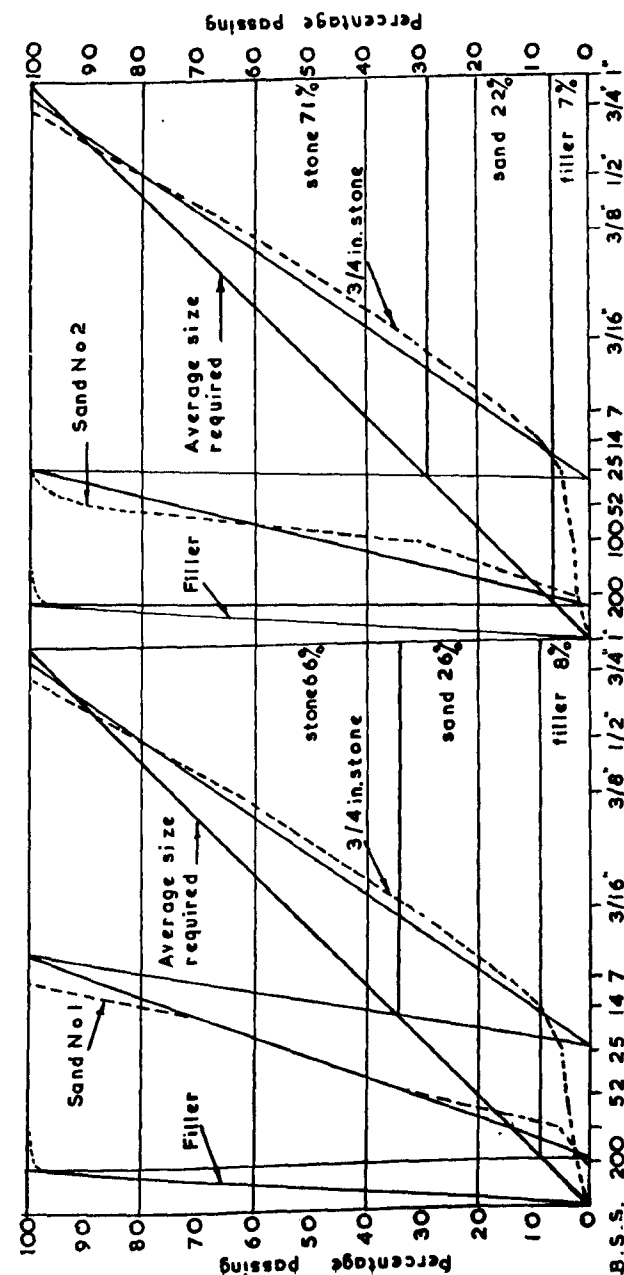


Fig. 19. Proportioning of aggregates

TABLE 7

Asphaltic concrete gradings.

Grading with Sand No. 1

	$\frac{3}{4}$ in. stone	Sand No. 1	Filler	Combined	Mean
Passing $\frac{3}{4}$ in.	100 66			100	95
„ $\frac{3}{8}$ in.	61.7 40.7			74.7	72.5
„ $\frac{3}{16}$ in.	31.5 20.8			54.8	53
„ No. 7	14.7 9.7	100 26		43.7	40.25
„ No. 14	8.3 5.5	72 18.7		32.2	35.5
„ No. 25	5.4 3.6	51.2 13.4		25.0	27.5
„ No. 52	4.1 2.7	30.7 8.0		18.7	21.0
„ No. 100	3.1 2.0	5.1 1.3	100 8	11.3	14.0
„ No. 200	2.2 1.4	0.7 0.2	96.4 7.7	9.3	8.0

Grading with Sand No. 2

	$\frac{3}{4}$ in. stone	Sand No. 2	Filler	Combined	Mean
Passing $\frac{3}{4}$ in.	100 71			100	95
„ $\frac{3}{8}$ in.	61.7 43.7			72.7	72.5
„ $\frac{3}{16}$ in.	31.5 22.4			51.4	53
„ No. 7	14.7 10.4			39.4	40.25
„ No. 14	8.3 5.9	100 22		34.9	35.5
„ No. 25	5.4 3.8	99.1 21.8		32.6	30
„ No. 52	4.1 2.9	79.9 17.6		27.5	24
„ No. 100	3.1 2.2	31.9 7.0	100 7	16.2	18
„ No. 200	2.2 1.6	3.3 0.7	96.4 6.7	9	8

10.2. Series J & JB

The aggregate grading used for this series approximates closely to that of the Fuller curve for aggregate passing the $\frac{3}{4}$ in. sieve. Sand No. 2 had not been used in the mixtures so far studied as the voids in this sand are high. In view of the small quantity required in this mixture, its use can be considered and its behaviour in the tests observed.

10.3. The results of the tests are plotted in Fig. 20. Two degrees of compactive effort were employed in order to assess the effects of additional compaction on the road. Series J was compacted using 25 blows of the standard tamper on each side of the sample while in Series JB, 50 blows on each side were used. In Series J, 5 per cent of air voids were obtained with 5.4 parts by weight of bitumen to 100 parts of aggregate, while in Series JB this degree of compaction could be obtained with 4.4 parts of bitumen. An examination of the Stability and Resistance curves shows that the additional compactive effort has the following results.

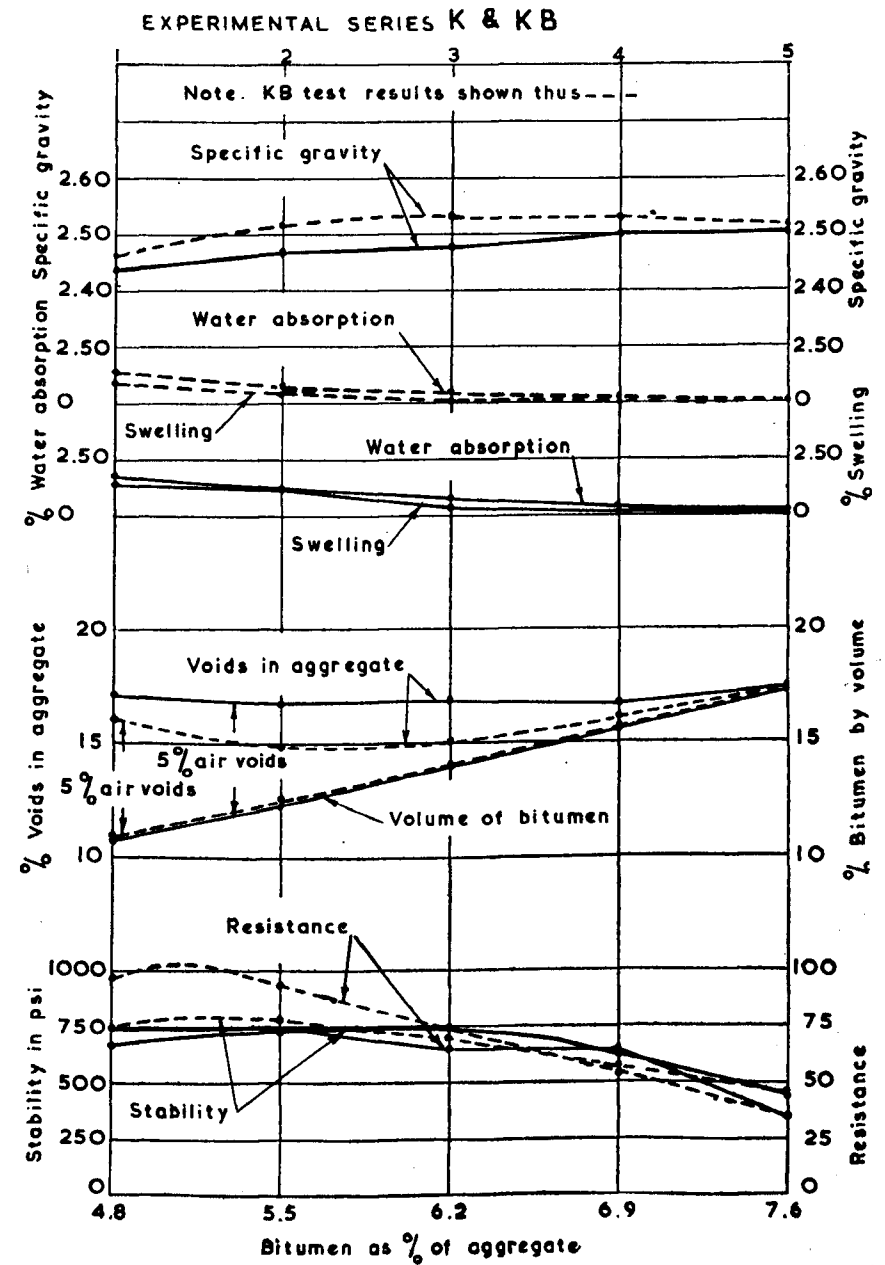
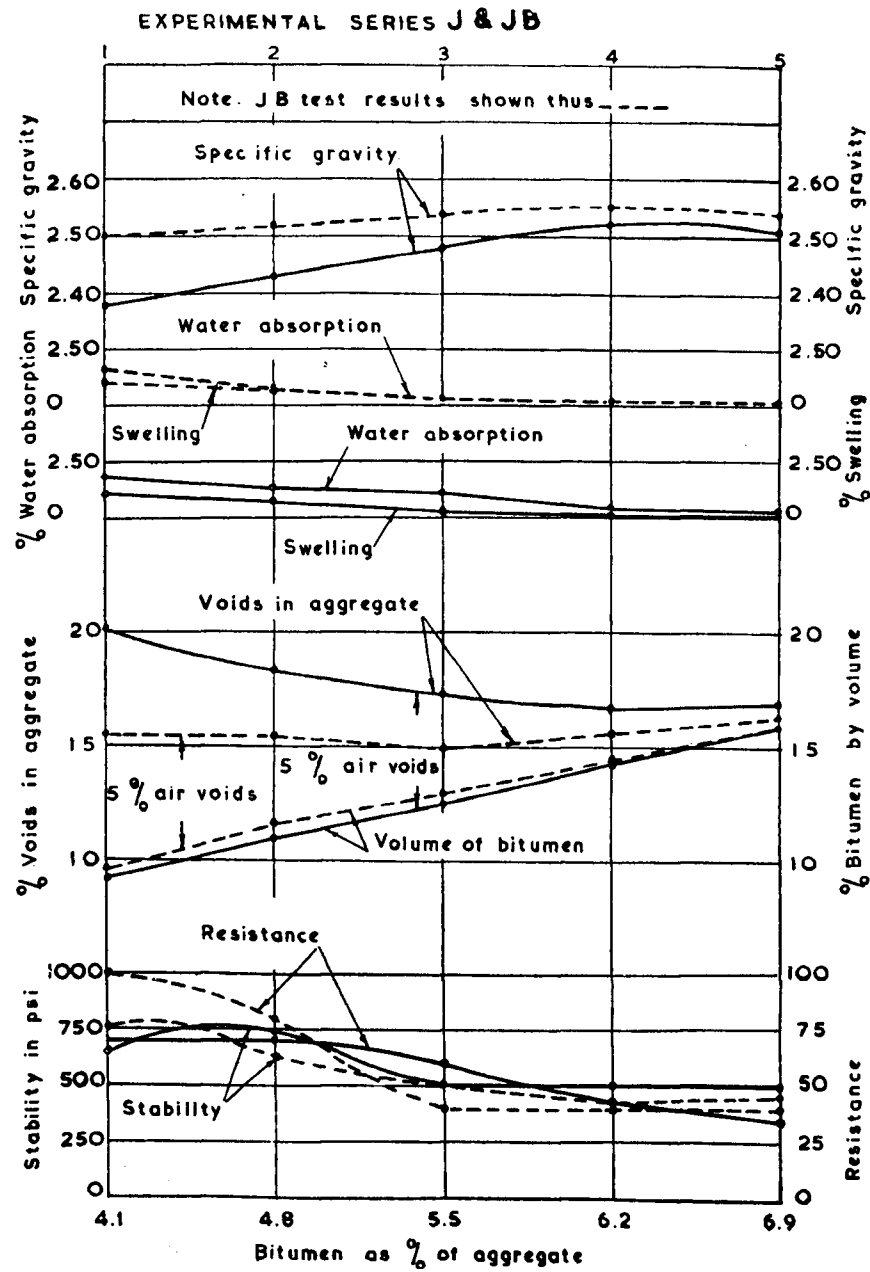
Lean mixtures	—	Stability	and	resistance	increased
Normal	„	—	„	„	reduced
Rich	„	—	„	„	unchanged.

Whilst a mixture of this grading having a bitumen content of about 5.5 per cent of the aggregate weight would be suitable for use as a binder, the reduction in stability following the additional compaction under traffic would render it unsuitable as a wearing surface.

10.4. Series K & KB

In Series K, Sand No. 2 is replaced by Sand No. 1 and the proportions have been calculated to give an aggregate grading recommended for an asphaltic concrete wearing surface. In Fig. 21 the test results on Series K and on Series KB, prepared with additional compactive effort, have been plotted. The bitumen range was changed from 4.1 - 6.9 parts per 100 to 4.8 - 7.6 parts per 100 as it was thought that the increase in sand content over that used in Series J would necessitate the use of additional bitumen. The plotted results show that, with standard compactive effort, the aggregate voids are reduced to a minimum of 16.5 per cent both in Series J and Series K but that, while in Series J 6.2 parts of bitumen are required to obtain this minimum, in the case of Series K only 5.5 parts per 100 are required. The superiority of Sand No. 1 is thus again demonstrated.

10.5. The values of stability and resistance to deformation are satisfactory and it will be seen that traffic compaction will result in no decrease in their values. A mixture having a



bitumen content of from 5.5 to 6.2 parts per 100 should give very satisfactory results.

10.6. Series L & LB

In Series L, Sand No. 2 is again used and the test results with standard and additional compactive effort are shown in Fig. 22. No satisfactory mixture can be obtained within the bitumen range employed, while the use of additional bitumen would result in a further lowering of the stability. 5 per cent air voids with standard compactive effort can be obtained by using 6.75 parts of bitumen per 100.

11. WORKING CONTROLS

11.1. After a decision has been made on the aggregate grading and the bitumen proportion, normal inspection methods should ensure that the grading and proportions are maintained. If aggregates come from different sources of supply and have to be blended, it may be essential to screen the dried aggregate and to specify the actual mixture in terms of weights to be delivered from each bin fed from the screens. For the highest class of work this is essential. In India it will usually be necessary for the asphalt surfacing contractor to provide and run his own quarry and crushing installation. In these circumstances it will often be possible to blend the crushed stone and the sand before they are dried and the necessity of an expensive gradation control and screening unit will be obviated.

11.2. It is often forgotten by inspecting staff that the primary requirement is an asphalt mixture with certain characteristics which may be specified while the exact composition is a secondary consideration. Compliance with the specification can and must be checked by rapid physical tests. Analysis of the mixture as a check on compliance with the specified proportions must also be carried out but unless a laboratory is available on the working site days may elapse before the results are known, while even a site laboratory will require several hours for a complete analysis. The physical characteristics which are capable of rapid determination by the methods described in this Paper are water absorption, swelling, specific gravity, stability and resistance to deformation. All these results can be given by a trained staff on the site within one hour of the material being mixed, while the water absorption and swelling results can be available in 30 minutes. These physical tests are the real working controls which will ensure that the mixture will have the characteristics essential to a satisfactory asphalt mixture.

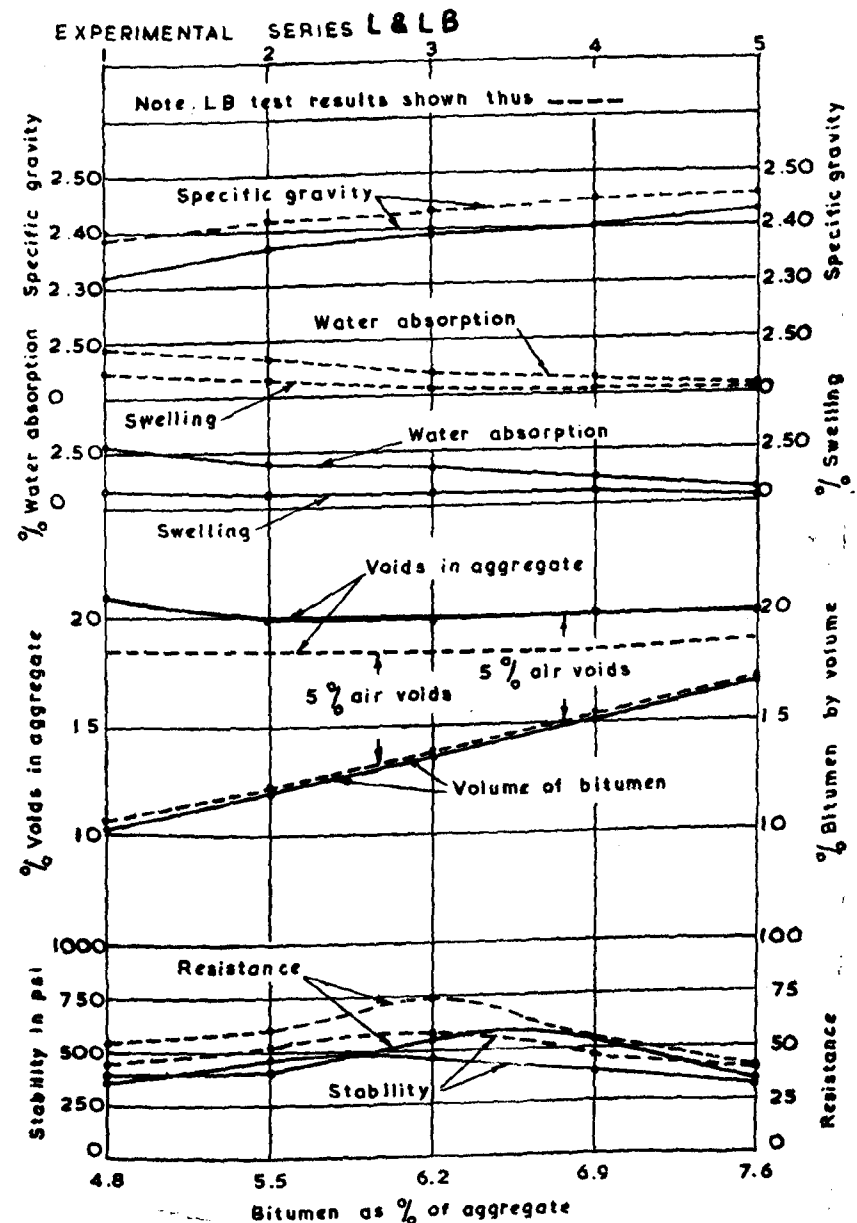


Fig. 22.

12. REVIEW OF RESULTS

12.1. The tests results quoted in this Paper give some indication of the wealth of information which can be obtained with comparatively simple apparatus. It may have come as a surprise to some engineers that there could be so much variation in the characteristics of a mixture of bitumen and aggregate. It has been shown that mixtures may have almost identical gradings and bitumen contents with entirely different physical characteristics; it has been shown that specification in terms of composition is not enough and that no mere formula will ensure a satisfactory mixture.

12.2. The information obtained from the test results may be summarised as follows:

(a) Water absorption.

Water absorption should be low especially if accompanied by swelling.

(b) Swelling.

Swelling is dangerous as it opens up the pores in the mixture and thereby increases the water absorption.

(c) Voids.

High and low limits are normally set for air voids. From 2—5 per cent is a reasonable range if the standard compaction described is used but samples should be checked for stability when prepared with additional compactive effort.

(d) Stability.

Stability may be high when the air voids are over 5 per cent and such mixtures may have to be used at places subjected to very heavy loads. An alternative is to use a hard bitumen at these locations. When the air voids in a mixture are low the stability is a function of the viscosity of the filler bitumen mixture.

(e) Stone.

Stone may be used for cheapening sheet asphalt but care must be taken to give adequate compaction or water absorption may be high. Stone used in an asphaltic concrete should be graded for density to achieve maximum economy in bitumen.

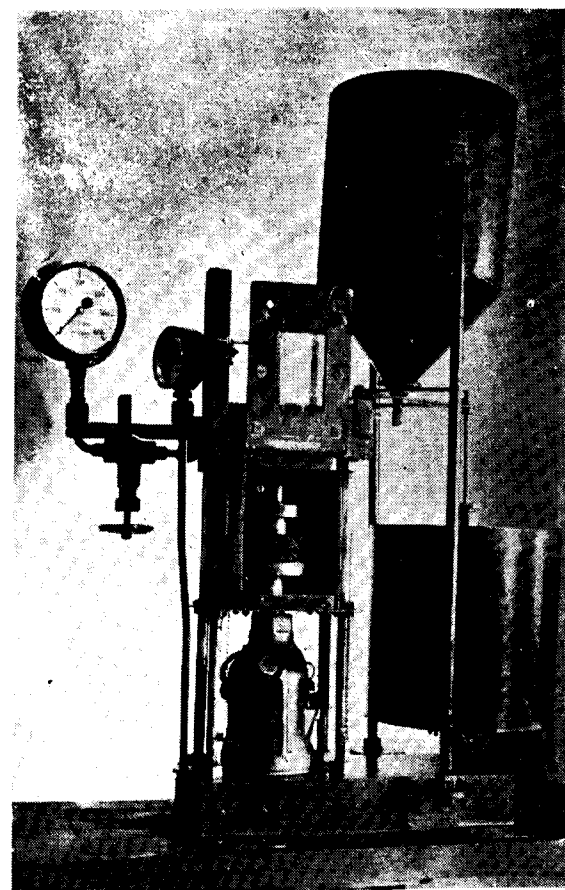


Fig. 23.

Taylor Stability Testing Machine.

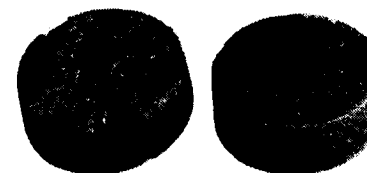


Fig. 24.

Stability Test Samples.

(f) Sand.

The restrictions placed by many specifications on sand grading to ensure a fine sand similar to that used by Clifford Richardson appear to be unnecessary. More dense and more economical mixtures can be prepared from a coarser, graded sand. Some sands may cause swelling. Although experimental results are not quoted here, it should be recorded that the swelling caused by Sand No. 1 was due to humus in the sand.

(g) Bitumen.

The stability of a mixture having low voids is related to the viscosity of the bitumen. The quantity of bitumen which can be used in an asphalt mixture is determined by the voids in the aggregate and has no relation to the surface area of the aggregate. It has not been possible to deal with the question of the optimum bitumen content for absorbent aggregates but it will be obvious that the determinations made by the methods described will only be true for freshly prepared mixtures. As bitumen becomes absorbed by the aggregate, the apparent specific gravity will remain unchanged but the air voids in the aggregate will increase and the stability will probably increase.

(h) Filler.

Fillers should be chosen carefully as the stability of an asphalt mixture having low voids is related to the viscosity of the filler-bitumen mixture. Even an excellent filler may not have beneficial results unless used in the correct proportion.

13. CONCLUSION

This Paper is intended to demonstrate that a certain amount of care is required in specifying asphalt mixtures if bitumen, which is now an indigenous product, is to be used successfully and economically. Trial and error may yield results but once hot mixed asphalt in India is accepted as a permanent surfacing, the quantities laid will be so great that any errors in mixture design will be financially disastrous and it is now that the foundations must be laid for a logical and scientific approach to the subject of specifying the asphaltic mixtures which will in the near future be carrying traffic throughout India.

REFERENCES

1. Soil Mechanics for Road Engineers, H.M.S.O.
 2. Broome, The Testing of Bituminous Mixtures.
 3. Taylor, Journal Society of Chemical Industry, 1932.
 4. British Standard 594 : 1950.
 5. Road Research, 1954, H.M.S.O.
 6. Road Research, 1956, H.M.S.O.
 7. Bituminous Construction, Barber-Greene Company.
 8. Swaminathan, Stability Testing of Asphalt Paving Mixtures, London University Thesis.
 9. Bituminous Road Construction, Burmah-Shell Oil Storage and Distributing Co. of India Ltd.
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A VIEW OF THE COMPLETED BRIDGE OVER HINDAN RIVER AT GHAZIABAD

Paper No. 201

"CONSTRUCTION OF PRESTRESSED CONCRETE SUPERSTRUCTURE FOR THE BRIDGE OVER RIVER HINDAN AT GHAZIABAD"

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SYNOPSIS

River Hindan lies between Delhi and Ghaziabad on a very important route connecting Delhi with Uttar Pradesh. The old bridge was a steel girder bridge 14 feet wide. It has been replaced by a prestressed concrete superstructure 24 feet wide over the existing piers which were already wide enough. Freyssinet type of prestressing has been used.

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The bridge has 5 spans 82 feet clear. The new superstructure consists of 3 simply supported prestressed concrete T-beams which were precast and launched. The existing steel girders were made use of for launching. The deck slab and cross beams were cast-in-situ and were transversely prestressed.

The Paper describes in detail the precasting of beams, their prestressing and launching into spans. The design of the mix and the experiments conducted in the field have been described. The outline of the design of the bridge has been given. The load test carried out in one span has also been described giving the observations recorded and inferences drawn.

1. INTRODUCTION

1.1. General

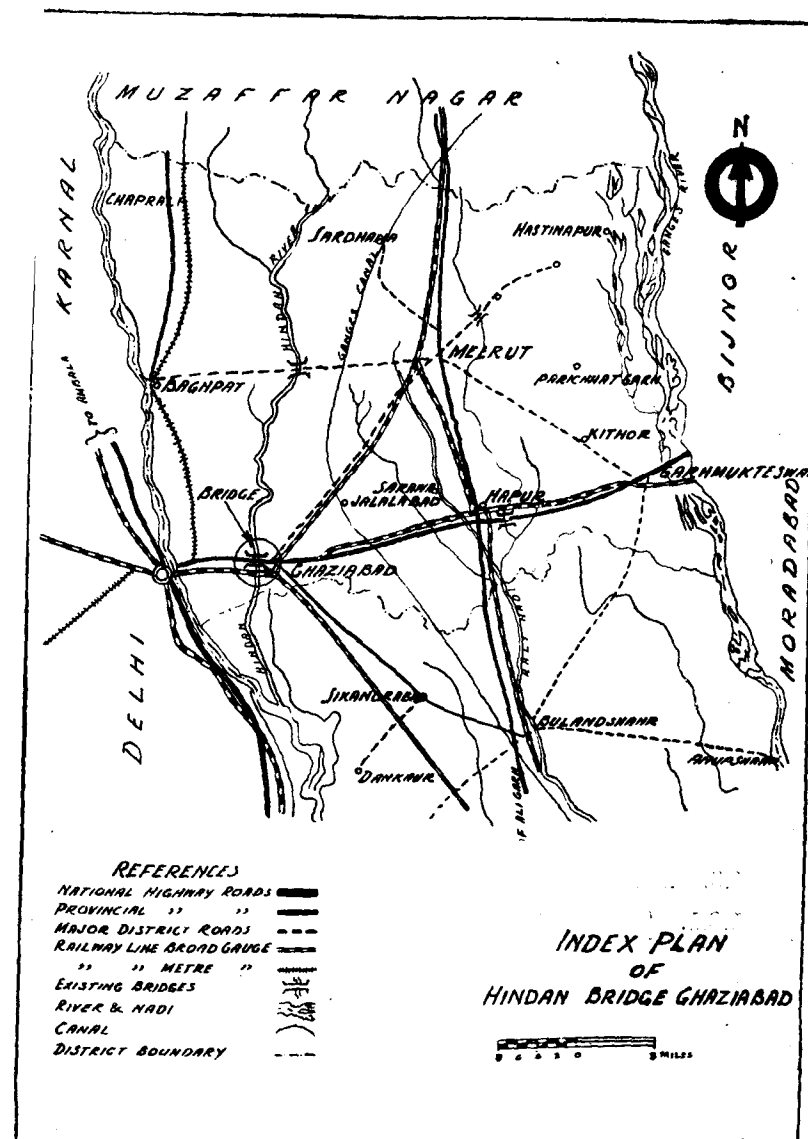
1.1.1. The Hindan Bridge lies in mile 11 of National Highway No. 24 between Delhi and Ghaziabad, in which length the highway is common with Delhi—Meerut Road and the Grand Trunk Road. It is, therefore, a very important bridge serving three main routes and carries a very heavy slow as well as fast traffic. According to a recent traffic census the average number of vehicles crossing the bridge daily were 984 trucks and buses, 384 cars, 178 carts and thelas and 167 tongas, ekkas and rickshaws. The average intensity of traffic on the bridge works out to 6244 tons per day, comprising of 5859 tons of automobile traffic and 385 tons of slow moving animal drawn vehicles.

1.1.2. The river Hindan takes off in Saharanpur district and flowing through Muzaffarnagar and Meerut districts meets the Yamuna in Bulandshahr district. This bridge lies about 16 miles upstream of the confluence of the river with Yamuna. The Index Plan shows the position of the bridge.

1.1.3. During dry weather the river carries very little water of its own. But 6 furlong downstream of the site of this bridge, there is a weir to feed the Hindan Canal, which irrigates the area in this locality. The weir gates are kept up throughout the dry season and so the depth of water at the bridge site is considerable during this part of the year also though the velocity is very little.

1.2. The Old Bridge

1.2.1. The old bridge was of through type steel truss girders over masonry piers, having 5 spans 92 feet centre to centre of piers. The width of roadway on the bridge was only 14 feet 6 inches which was inadequate for two-way traffic. As a result vehicles from the other side had to wait while traffic from one side passed over the bridge and occasions were not rare when



vehicles from both sides would enter the bridge and then one side would have to back after quite long arguments. The need to widen the bridge was, therefore, imperative to remove this bottle-neck in the free movement of the traffic on this important National Highway.

1.2.2. The masonry of the piers and abutment is in very sound condition and the length available at their top was also over 50 feet; hence it was decided to use the old substructure and to change the superstructure only to widen the bridge.

1.2.3. But this being a very old bridge no records existed of the foundations, which were required to decide the type of new superstructure. Investigation of foundations was, therefore, taken up. For this a pit approximately 10 feet diameter was made by the side of the first pier on Ghaziabad side. The pit was dug upto 8 feet below the bed level and at this depth a 4 feet thick block kankar pitching floor was found. This was also pierced through and sinking below it was done with 2½ feet diameter hume pipe as ordinary excavation was not possible. This was carried upto 6 feet below the bottom of the kankar floor and pucca work of the pier was met upto this level, while after this depth no pucca work was met. This led to the conclusion that the pier was resting on open foundations. The 4 feet thick flooring exists throughout the length of the bridge and also extends 15 feet beyond the nosing of the piers on the upstream side and about 25 feet on the downstream side. The boring chart is given in Plate I.

1.2.4. The details such as width and length of foundation, however, could not be found out but it can be worked out that the increase in bearing pressure at the base due to new superstructure will not be more than 20 per cent and so is covered by the safety factor which may have been adopted for the original design of the bridge.

1.3. New Bridge

1.3.1. The new bridge was to have a roadway 24 ft wide between curbs providing 2 lanes of traffic and was to be designed for National Highway Loading of I. R. C. Bridge Code.

1.3.2. There were two alternatives in which the widening could be effected, viz.,

(a) Shifting the existing steel trusses to have a clear distance of 24 feet between them and to carry out necessary strengthening, or

(b) replacing the old structure completely with a new superstructure in reinforced or prestressed concrete.

1.3.3. The alternative (a) was abandoned for the following reasons :

- (i) If this was adopted, the cross girders would have to be replaced and lattice girders would have to be strengthened necessitating changing most of its members. It was estimated that all this would require about 370 tons of new structural steel which would be difficult to arrange in the present day scarcity conditions.
- (ii) The pieces of old steel which would have been rejected (about 96 tons) would be completely useless and could only be sold as scrap iron.
- (iii) The existing steel structure was quite old and it was not considered desirable to reuse it on such an important highway bridge carrying such heavy traffic.
- (iv) These old steel girders could, in case a new superstructure was adopted for this bridge, be reused in construction of small bridges on hill roads where traffic is not so heavy and for which the department was facing a lot of difficulty in arranging steel. It is convenient to use steel bridges in hills due to the problems of transport and erection of stagings.
- (v) It was found that a concrete superstructure would work out cheaper than alternative (a) specially because old steel girders could be used elsewhere and credit allowed for them to this estimate.

1.3.4. It was, therefore, decided to have a completely new superstructure in concrete.

1.3.5. The main considerations in deciding whether reinforced concrete or prestressed concrete superstructure should be used were cost and dead load of the structure. It was found that a prestressed concrete superstructure was lighter and cheaper and was, therefore, adopted.

2. DESIGN FEATURES

2.1. Main Dimensions

2.1.1. In the new superstructure each span consists of three precast prestressed longitudinal T-beams resting on cast steel bearings placed at a distance of 10 feet 6 inches centre to centre. The length of the beams end to end is 91 feet 6 inches

for central three spans and 93 feet 6 inches for the end spans, the length between centres of bearings being 3 feet 2 inches less. The length of beams in end spans is 2 feet longer as the bearings on the abutment have been placed at the same position as they were for the steel trusses. There is a gap of 2 inches between two consecutive beams.

2.1.2. The depth of the beams is 5 feet 10 inches including the deck slab. The web of the beams is 6 inches thick in the central 53 feet length and gradually increases to 12 inches at the ends. The width of the bottom flange is 1 foot 6 inches and that of the top is 5 feet. There are 6 cross beams in each span, the end ones being 10 inches wide and the rest 8 inches.

2.1.3. The deck slab consists of two portions, one the flange of the beam and the other the cast-in-situ slab 7 inches thick between the flanges of two beams. These two parts are held together by cross-prestressing.

2.1.4. The roadway is 24 feet wide clear between curbs which are 5 inches wide and 9 inches high. The wearing coat is 3 inches thick cement concrete. For the railing U.P.P.W.D., standard design of R.C.C. railing for major bridges has been adopted. It consists of 3 elliptical shaped horizontal rails supported on vertical posts 5 feet 10 inches centre to centre.

2.1.5. A section of the superstructure is shown in Fig. 1.

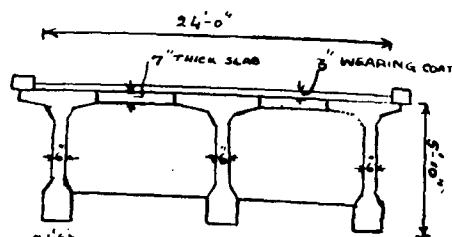


Fig. 1. Section of superstructure

2.1.6. The prestressing cables are made up of 12 numbers of 7 mm. diameter high tensile steel wires arranged round a helical spring and encased in a steel tube. These are ten in number for longitudinal prestressing, while for transverse prestressing, three numbers have been used in each cross-beam and one cable at 4 ft 5 inches interval is used in deck slab.

2.2. Loads and Specifications

2.2.1. The structure has been designed to carry two traffic lanes of class A or one lane of class AA loading specified by the

Indian Roads Congress, whichever gives maximum stresses. The design conforms to Indian Standard Code of Practice for Plain and Reinforced Concrete (I.S. 456-1953), the Standard Specifications and Code of Practice for Road Bridges in India, Draft Criteria for Prestressed Concrete Bridges and Provisional Specifications for Cast Steel Bearings issued by the Indian Roads Congress.

2.3. Ultimate and Working Stresses

2.3.1. The following values of ultimate and working stresses of various materials have been adopted in the design:

(a) Concrete

Average strength of 4 inch by 4 inch test cube after 28 days	... 5360 lb per sq. in.
Maximum permissible temporary stress	... 1900 lb per sq. in.
Maximum permissible working stress	... 1600 lb per sq. in.
Maximum permissible principal tensile stress	... 115 lb per sq. in.
Maximum permissible working tensile stress	... '0'

(b) Steel

Ultimate strength according to test certificates	... 2,27,000 lb per sq. in.
0.1 per cent proof stress	... 2,05,000 lb per sq. in.
Maximum permissible temporary stress at jacks	... 2,05,000 lb per sq. in.
Maximum permissible stress before losses due to creep and shrinkage 0.6 of 227000	... 1,36,000 lb per sq. in.

2.4. Deck Slab

2.4.1. The deck slab is designed as continuous along the supporting beams by drawing influence lines for haunched sections and calculating the maximum moments. It was found that class AA tracked vehicle governs the design.

2.4.2. The slab has been prestressed with cables consisting of 12 number 7 mm. high tensile wires placed at intervals of 53 inches along the bridge. The cables are placed with their centre of gravity $4\frac{7}{8}$ inches below the upper edge at centre line of span and 3 inches below at the end of haunches.

2.4.3. The cross cables were stressed to 86,500 lb per cable.

2.4.4. The maximum and minimum stresses in concrete work out as given in Table 1.

TABLE I
Stresses in deck slab

Position	Maximum stress in lbs per sq. inch.	Minimum stress in lbs per sq. inch.
1. Mid-span	+ 454	+ 22
2. Supports	+ 321	+ 9
3. Ends of haunches :		
(a) Positive moment	+ 462	+ 14
(b) Negative moment	+ 5	+ 471

2.4.5. The mild steel reinforcement provided arbitrarily in the deck slab consists of $\frac{3}{8}$ in. diameter bars placed longitudinally 6 inches centre to centre at the bottom throughout the slab and 5 inches centre to centre at the top over cross beams. $\frac{3}{8}$ in. diameter bars have been provided at the bottom in the transverse direction also.

2.5. Main Beams

2.5.1. Beams as already stated have a T-section, the dimensions of which in the centre of span are given in Fig. 2.

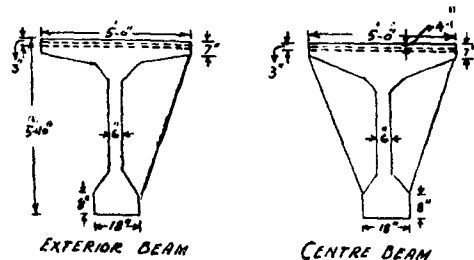


Fig. 2.

2.5.2. The total dead load bending moment for one beam works out to 998 ft tons. For live load it was found that class AA loading gives the maximum stresses in beams.

2.5.3. The distribution of live load to various beams has been worked out assuming a linear distribution.

2.5.4. According to I.R.C. standard specification, the most eccentric position of the vehicle to be taken into consideration for the design is the one given in Fig. 3.

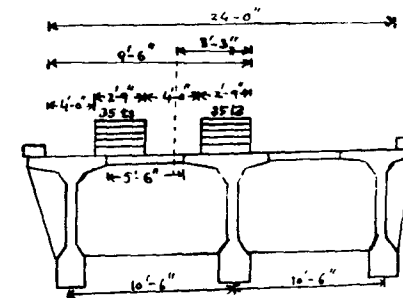


Fig. 3. Distribution of live load

2.5.5. Maximum eccentricity of resultant of load is 3 ft 3 in. The ratio length : width of the bridge and the dimensions of the beams allow the cross beams to be considered as indefinitely rigid in comparison to the main beams.

2.5.6. In the case sketched above, the consequence of this is that the left beam will carry the following portion of the live load :

$$\frac{P}{3} + \frac{3.05 \times 10.5}{2 \times (10.5)^2} \times P = 0.333P + 0.155P = 0.488 P.$$

The load on other beams will be :

$$\text{Central beam} = 0.333 P$$

$$\text{Outer light beam} = 0.178 P$$

2.5.7. The live load bending moment for class AA tracked vehicle works out to 772 feet tons.

2.5.8. Since the beams have been precast and then launched they are subject to varying loads during each stage of handling. They were tested for the following cases :

- (a) Each beam supporting its own weight, when it was removed from its casting bed and taken to span.

- (b) Each beam as at (a) plus all dead load when the cast-in-situ slab, wearing coat and curb has been cast.
- (c) Each beam as at (b) plus full live load.

2.5.9. All beams were prestressed by means of ten cables each consisting of 12 wires of 7 mm. diameter. The sectional area of cable is 0.72 sq. inches.

2.5.10. All cables were initially stressed to 136,000 lb per sq. inch at the centre of the span. Due to friction the corresponding stress at the anchoring blocks worked out to 171,000 lb per sq. inch and allowing for slippage during locking the stress at jacks would increase to 181,000 lb per sq. inch.

The losses due to creep in concrete and steel and shrinkage in concrete would be 24,900 lb per sq. inch and effective prestressing force at centre of beam would be $0.72 (136000 - 24900) = 82000$ lb per sq. inch.

2.5.11. The stresses at various stages of construction are given in Table 2.

TABLE 2
Stresses in beam

Stage	Upper edge lb per sq. in.	Lower edge lb per sq. in.
(i) Dead weight of beam	... + 550	- 1100
(ii) First prestress (without losses) 7 cables	... - 490	+ 2830
(iii) Due to (i) & (ii) combined	... + 60	+ 1730
(iv) First prestress (effective) cables	... - 410	+ 2365
(v) Due to (i) & (iv) combined	... + 140	+ 1265
(vi) Due to cast-in-situ concrete	... + 208	- 380
(vii) Due to (v) & (vi) combined	... + 348	+ 885
(viii) Due to wearing coat curb & railing	... + 181	- 420
(ix) Due to first prestress & total dead load (vii) + (viii)	... + 529	+ 465

Stage	Upper edge lb per sq. in.	Lower edge lb. per sq. in.
(x)* Due to loss * in the 7 cables previously prestressed due to stressing of remaining 3 cables	... + 19	- 105
(xi) Due to second prestress (3 cables)	... - 130	+ 905
(xii) Stress due to total prestress & total dead load (ix)+(x)+(xi)	... + 418	+ 1285
(xiii) Due to live load	... + 560	- 1257
(xiv) Total stress due to all loads (xii)+(xiii)	... + 978	+ 8

2.5.12. The values of maximum reaction at the bearings and maximum shear force are given below :

Position	Maximum reaction at the bearings	Value in tons
(a) Due to live load—		
(i) Vehicle placed symmetrically over the centre beam—for centre beam	...	44.4
(ii) Vehicles placed with maximum eccentricity— for exterior beam	...	29.2
(b) Dead load for all beams	...	46.6
Total load = 44.4 + 46.6	...	91.0
Maximum shear force		
(a) Due to live load—		
(i) Centre beam - vehicle placed symmetrically	...	40.6
(ii) Exterior beam	...	29.4
(b) Dead load	...	45.2
Total shear (maximum for central beam)	...	85.6

* The value of this loss is determined by the elastic deformation of concrete only, as the effect of the losses due to the deferred deformation have already been taken into account.¹

2.5.13. The maximum shear stress occurs at the lower edge of upper triangular haunches, where its value comes to 92 lb per sq. inch. Combined with the compressive force due to prestress the principal tensile stress works out to 28 lb per sq. inch. At the upper edge of lower triangular haunches these values are 50 lb per sq. inch and 6 lb per sq. inch respectively.

2.5.14. **Mild steel reinforcement.** There are 10 bars $\frac{1}{4}$ inch diameter and 4 bars $\frac{3}{8}$ inch diameter longitudinally. Stirrups $\frac{3}{8}$ inch diameter of different shapes to suit the section of beam have been provided at 1 foot 8 inches centre to centre in the central 53 feet length and 1 foot centre to centre in the remaining length at the ends. The exterior beams have additional stirrups $\frac{1}{2}$ inch diameter 8 inches centre to centre throughout. Stirrups have also been provided to reinforce the concrete around the prestressing cones.

2.6. Cross Beams

2.6.1. The main effect of cross beams is to distribute the live load linearly between the main beams.

2.6.2. The cross beams are of T-section with width of flange 8 inches, depth of flange 7 inches and a height of 62 inches. The thickness of web is 8 inches for all beams except those at the ends for which it is 10 inches.

2.6.3. Two cables, similar to deck slab, have been provided in the web of cross beams.

2.6.4. The mild steel reinforcement in cross beams consists of 6 numbers $\frac{3}{8}$ inch diameter bars and $\frac{3}{8}$ inch diameter stirrups have been provided 12 inches apart.

2.7. End Blocks

2.7.1. The end blocks have been designed according to the method outlined in "Prestressed Concrete" by Y. Guyon.

2.8. Expansion Joints

Expansion joints of the type shown in Fig. 4 have been provided in the deck slab at the piers and abutments. They consist of 2 angles 4 inches by 3 inches by $\frac{3}{8}$ inch anchored in concrete by means of $\frac{3}{4}$ inch bars welded to them at 18 inches centre to centre. On the top of the angles $\frac{3}{8}$ inch plates have been screwed with holes staggered 6 inches centre to centre. The

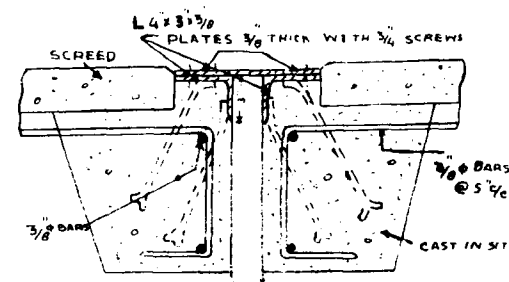


Fig. 4. Expansion joint

plate on one angle projects on to the other angle as shown in the Figure while the plate on the other angle is shorter in width.

2.9. Bearings

2.9.1. The bearings are designed for a maximum load of 91 tons and a minimum load of 45 tons. The horizontal force from braking vehicle is 14 tons. The bearings are made of cast steel.

2.9.2. Rocker bearings consist of two plates with transverse lugs in the middle, the bottom one having a curved face with 10 inches radius and the top one is plane.

2.9.3. The dimensions of the lower plate is $19\frac{1}{4}$ inches by 21 inches and of upper plate is 17 inches by 18 inches.

2.9.4. The maximum intensity of stress between steel plate and concrete is 735 lb per sq. inch.

2.9.5. Roller bearings consist of two plates with a roller in between. The upper plate is 17 inches by 18 inches and $2\frac{5}{8}$ inches thick and the lower plate is 17 inches by 21 inches and $2\frac{5}{8}$ inches thick.

2.9.6. The roller faces have a diameter of 10 inches. To save steel the roller has not been made as a cylinder but has vertical sides and is only 3 inches wide in the middle.

3. MATERIALS & MIX

3.1. Concrete

3.1.1. The samples of coarse and fine aggregates submitted by the firm were tested in the P.W.D. Research Laboratory at Lucknow and field tests were carried out to see that the grading actually used conformed to the samples.

3.1.2. **Coarse Aggregate.** The coarse aggregate consisted of crushed granite obtained from Delhi. Two sizes—nominal $\frac{3}{4}$ inch and $\frac{3}{8}$ inch were obtained. The physical properties of the two sizes are given in Table 3.

3.1.3. **Fine Aggregates.** The fine aggregates consisted of natural coarse sand obtained from Badarpur near Delhi. Physical properties are given in Table 3.

TABLE 3
Physical properties of aggregates

	Coarse aggregate		Fine aggregate. per cent
	$\frac{3}{4}$ inch size. per cent	$\frac{3}{8}$ inch size. per cent	
1. Sieve Analysis			
Retained on $1\frac{1}{2}$ inch mesh sieve	Nil	Nil	Nil
$\frac{3}{4}$ " " "	4.1	Nil	Nil
$\frac{3}{8}$ " " "	96.4	35.08	Nil
$\frac{3}{16}$ " " "	99.88	75.60	1
8 " " "	100.00	91.74	2.0
16 " " "	100.00	98.64	17.2
30 " " "	100.00	99.28	63.2
50 " " "	100.00	90.60	93.6
100 " " "	100.00	99.61	98.0
Fineness modulus	7.0038	5.9955	2.725

3.2. Cement

3.2.1. Cement used was ordinary portland cement.

3.3. Mix Used

3.3.1. The mix used for beams, slabs etc. was designed to have a minimum strength of 5360 lb per sq. inch after 28 days. The following mix was tested in the P.W.D. Laboratory and the average strength of 4 inches by 4 inches by 4 inches laboratory

cubes worked out to 5229 lb per sq. inch and 7250 lb per sq. inch after 7 and 28 days respectively.

$\frac{3}{4}$ inch aggregate—2.25 cu. ft.

$\frac{3}{8}$ inch aggregate—1.25 cu. ft.

Fine aggregate—1.25 cu. ft.

Cement—1 bag.

Water—4.5 gallons

W/C ratio—0.4 by weight.

This mix works out to approximately 1 : 1 : 2.8 by volume.

3.4. High Tensile Steel

3.4.1. The high tensile steel was tested in the laboratory of Roorkee University. The results were as follows :

TABLE 4
Results of tests on high tensile steel

Sample No.	Diameter in inches.	Area of cross section in sq. inches.	Ultimate load in tons.	Ultimate tensile strength in tons per sq. inch.
1.	0.278	0.061	6.17	101.0
2.	0.279	0.0612	6.22	101.5
3.	0.279	0.0612	6.22	101.5

3.5. Mild Steel

All mild steel used had been tested in the laboratory of Roorkee University and found to have a strength of over 27 tons per sq. inch intension.

4. CONSTRUCTION OF BEAMS

4.1. General

4.1.1. All the beams were precast and prestressed before launching in position. As the road embankment on both sides of the bridge is very high, the precasting yard had to be on the road and naturally much space was not available here. Four casting beds were made on the road embankment and the steel yard was on the guide bank. The assembling of the cables was done on the

road side on a slightly lower level. The general arrangement of casting yard has been shown in Plate II. It will be seen that more than 7 beams could not be stored in this yard at a time and so launching schedule had to be so arranged that more than this number never accumulated in the yard.

4.1.2. Before the beams could be concreted the end blocks had to be cast and cables assembled.

4.2. End Blocks

4.2.1. End blocks 2 feet 8 inches long were cast separately. The end blocks were heavily reinforced, the quantity of mild steel used being 0.7 per cent by volume. A slightly richer mix was used being:

1 : 0.8 : 2.6 by volume, water cement ratio being 0.4 by weight.

4.2.2. Timber formwork was used for casting these blocks. The female cones of the Freyssinet anchorages were fixed to the formwork to proper inclination by means of screws on their faces. G. I. pipes $1\frac{3}{4}$ in. diameter were provided to make holes in the end blocks for cables. The arrangement of fixing this cone is shown in Fig. 5. The junction of pipe and cone was sealed with adhesive tape to make it water tight to prevent the escape of mortar.

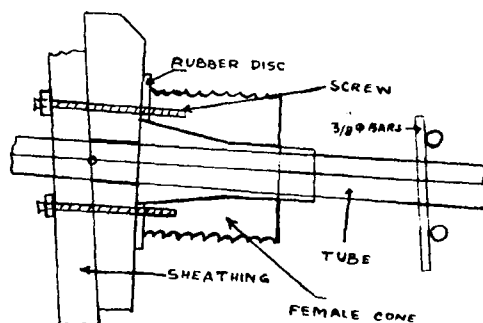


Fig. 5. Fixing of female cone

4.2.3. At the bottom of the end blocks mould was placed in the frame work which had inserts for the bolts which fix the upper plate of the bearings to the beam. These frames were prepared in advance with the help of a steel template obtained from the Ordnance Factory, Muradnagar (who manufactured the bearings)

so that the distance apart of the inserts be the same as in the bearings plate. The frame and inserts are shown in Fig. 6.

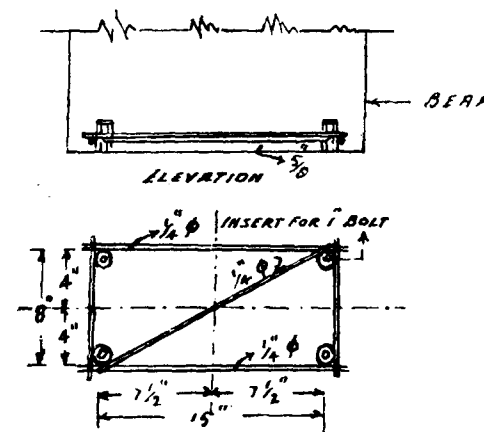


Fig. 6. Inserts for bearing

4.2.4. The mild steel reinforcement was first placed in position keeping one side of the formwork open, which was closed after the reinforcement had been finally checked. As the concrete near the female cones is subjected to greater stresses, spiral reinforcement is provided just in front of the cones. The mould was treated inside with mould oil.

4.2.5. A timber platform was erected on all four sides of the mould but not touching it for use by the men who would pour concrete and handle the immersion vibrator. The vibrator engine was also kept on this platform.

4.2.6. Three vibrators were used while concreting end blocks. Two electric form vibrators were fixed on the timber platform supporting the mould on either side and one immersion vibrator was used from the top. The form vibrators were switched on shortly after the concrete pouring was started and one or both switched on and off as found necessary. The immersion vibrator was used continuously. During concreting the formwork was carefully watched for any leaking joints in the timber which if noticed were plugged with wet paper.

4.2.7. The G.I. pipes which had been greased before concreting were rotated during concreting and this rotating was done occasionally during the next 4 hours or so after which they were taken out.

4.2.8. The side forms were removed the next day usually after about 18 to 24 hours and curing started by putting wet gunny bags all round which were kept wet by sprinkling water.

4.2.9. The end blocks for the three beams were slightly different from one another. In the centre beam end block no anchorage cones were required for transverse cables as in the end blocks for exterior beams.

4.2.10. The quantity of concrete in one end block was 34 cu. ft. with 117 lb of mild steel reinforcement. The weight of one end block was 2.25 tons approximately.

4.3. Making of Freyssinet Cables

4.3.1. The cables consisted of 12 high tensile steel wires 7 mm. (0.61 inch) diameter enclosed in a metal sheet tube. To keep the wires in position a core helix of G. I. wire was provided inside the cable. A section of the cable is shown in Fig. 7.

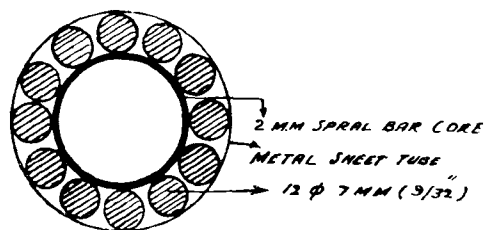


Fig. 7

4.3.2. The outer tube was made of a steel sheet, 15 cm. wide and 0.3 mm. thick, which was imported in rolls each weighing about 55 kg (121 lb). The weight of the sheet was 0.35 kg. per meter (0.23 lb per foot). The tube was made by means of a machine into which the sheet was fed at one end and pulled out from the other. As it passed between a number of rollers it was rolled into a tube $1\frac{3}{4}$ inches diameter with a water-tight joint. The machine could be worked by an electric motor but at this bridge it was operated manually by means of a crab winch.

4.3.3. The inner core helix was made by spinning a close spring of 26 mm. diameter out of a 14 gauge galvanised iron wire and then stretching it to about 25 times its compressed length.

4.3.4. The wires, tube and the helix were cut to required size. The tube was exactly equal in length to the distance between the holes of the end blocks measured along the final position of the cable. The wires were kept 5 feet longer than the length

of the cable in the beam to fix prestressing jack and the helix also of the same length.

4.3.5. To assemble a cable, the tube was laid on a table erected for the purpose. The 12 wires with the helix in between were first passed through a metal plate which had holes drilled in it and acted as guide and then pulled inside the tube by means of a thin steel rope.

4.3.6. Before the cables were fixed in the mould for the beams they were tested for leakage by passing water through them to make sure that they were watertight. Any leakage found was repaired by adhesive tape. Photo 1 shows such a repair

4.4. Shuttering of the Beams

4.4.1. Two concrete platforms 1 foot 8 inches wide, level at the top and longer than the length of the beam were made to form the casting bed of the beams. Across this platform wooden ballies were placed at 2 feet intervals and to these were nailed 2 inches thick wooden planks which formed the bottom shuttering of the beam. The bottom shuttering was removable in a length of 4 feet near both ends to allow for the jacks to be placed when lifting the beam. After the bottom shuttering had been fixed, the end blocks were placed at the two ends, the distance between them being accurately adjusted. Now the erection of one side form was taken up. These forms were in units of about 4 feet length and were made of timber planking with steel sheet facing. Originally it was not intended to use any sheet facing but as the timber was not of good quality this type of facing was adopted. To handle these forms and place them in position a gantry was used which moved on rail track laid on each side of the casting platform. This gantry was also used to place the end blocks in position. (Photo 2.)

4.4.2. Rubber packing was placed in the joint between the bottom form and the side form to make it watertight. After the form had been placed on full one side the cables were laid on the bottom form and mild steel reinforcement was fixed in position. To have required cover on mild steel reinforcement circular concrete spacers as shown in Fig. 8 were used.

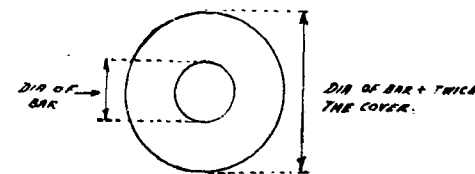


Fig. 8

4.4.3. The cables were now arranged. Six cables which passed through holes in the end blocks were first taken up. The junction of the tube with the end block was made watertight by means of adhesive tape. These cables were tied to the mild steel reinforcement at the bottom in the length where they were to run straight and hung between these points and the end blocks in the form of a catenary. The four cables which were to project out of the upper flange were also placed in position, their ends being bound temporarily to the mild steel stirrups. Photo 3 shows the cables in position.

4.4.4. After the cables were finally fixed in position the remaining mild steel reinforcement in the web end bottom flange was placed and then the other side form was fixed.

4.4.5. Concrete spacers were provided between the shuttering of the web and the two side forms were tightened together by means of form clamps as shown in Fig. 9.

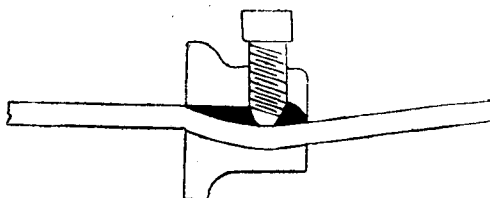


Fig. 9. Detail of form clamp

These clamps were fixed to $3/8$ inch diameter bars passing through the shuttering. They were tightened by means of a clamp key consisting of two parts, one of which was fixed to the bar and the other moved by a handle to tighten the form clamp. After the clamp had reached the desired position it was fixed to the bar by tightening the screw provided in it. Three rows of form clamps were provided—one at the top at a level higher than the top of the beam, one in the centre and one at the bottom below the bottom shuttering. Wooden struts were fixed at the top to keep proper distance between edges of the form for the upper flange. The two sides forms were then checked with a theodolite and all lines made dead straight and level.

4.4.6. The mild steel reinforcement in the top flange was then placed in position and tied up. The Freyssinet anchorage female cones for cables No. 7 to 10 were fixed with the help of wooden boxes fixed to the ballies at the top as in the case of end blocks.

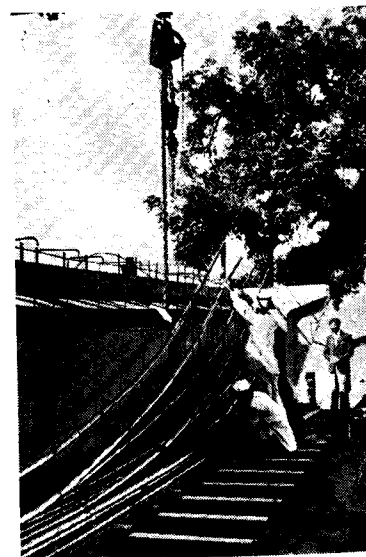


Photo 1.
Repair of leakage in tubes by
adhesive tape



Photo 2.
Placing end block in position

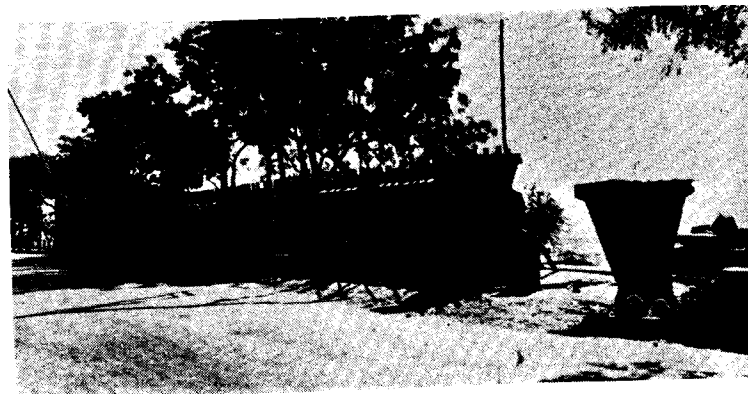


Photo 3. Cables fixed in position

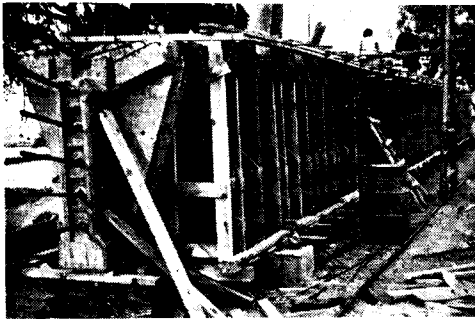


Photo 4.
Completed
shuttering of
beam



Photo 7.
Prestressing in progress

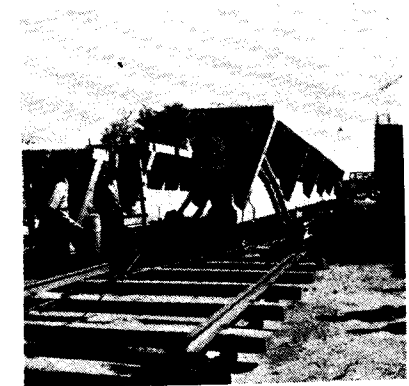


Photo 8.
Rolling of beam on rails

Photo 5.
Concreting—
gantry with
vibrators etc.

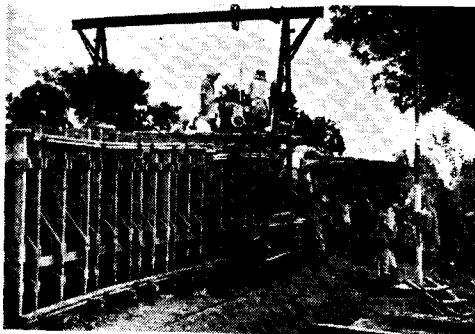
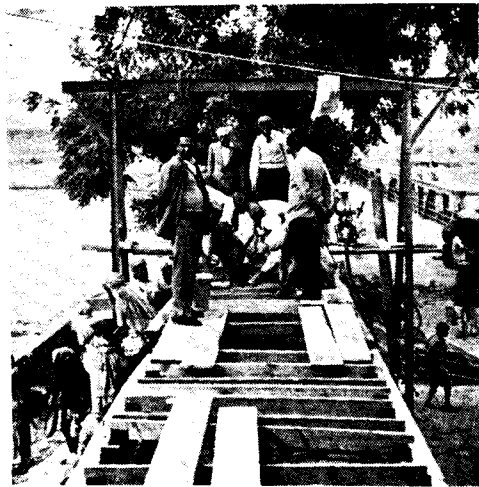


Photo 6.
Concreting
of beam

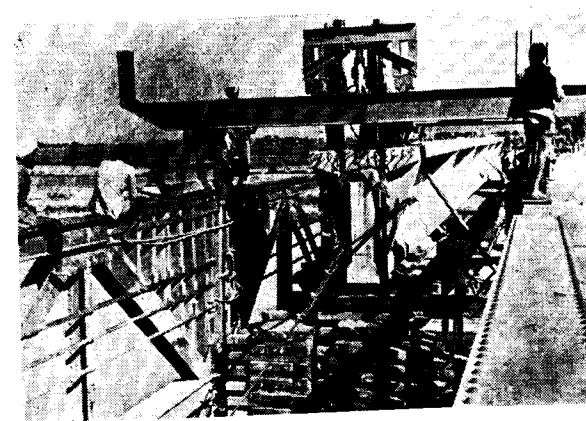


Photo 9.
Beam on top trolley being pulled



Photo 10.
Close view of
Freyssinet jack



Photo 11.
Close view of
hydraulic pump

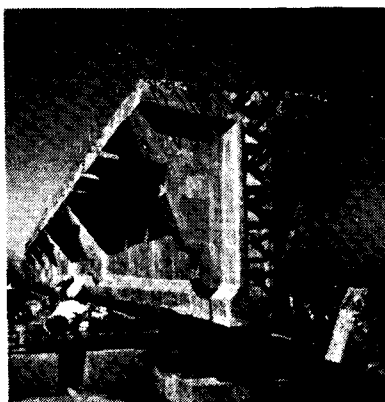


Photo 12.
Side rolling in
casting yard

4.4.7. Through holes in the side form of the upper flange and that of the cross beams, galvanised iron pipes were passed to form conduits for the cables required for cross prestressing in deck slab and cross beams respectively. Grease was applied to these pipes as well as the form work.

4.4.8. In the exterior beams wooden boxes were fixed at the ends of the holes for cables for cross prestressing to provide for the anchorage cones to be fixed later on.

4.4.9. It took 6 carpenters, 2 blacksmiths, 4 khalasis and 6 beldars about 4 days to erect the mould for one beam and fix all reinforcement and cables in position.

4.4.10. Photo 4 shows the complete shuttering.

4.5. Concreting of Beams

4.5.1. A day before the concreting was scheduled to be done all work on the shuttering was completed. A final check up was given just before starting the concreting. The equipment used consisted of one mixer of 9 cu. ft. capacity, two electric form vibrators and two immersion vibrators.

4.5.2. The two form vibrators were fixed to the battens at the bottom of the moulds. Later on, however, it was found that more effective vibration was obtained by fixing the form vibrators to the bottom of the shuttering in an inverted position. These vibrators were moved forward as concreting proceeded. The engines of the immersion vibrators were placed on a platform suspended from the gantry and could therefore move easily with the concreting. The labour pouring concrete into the mould and the persons holding the immersion vibrators also stood on this platform as can be seen in Photos 5 and 6.

4.5.3. Batching of aggregates was done by volume by batching boxes, the mix being as specified in para. 3.3.1. One box was made for each size of aggregate and its volume was a multiple of 1 bag of cement which was measured in terms of bags. Water was poured in the mixer by a gallon measure.

4.5.4. Slump test was performed on the mix obtained and was found to be $1\frac{1}{2}$ inch.

4.5.5. Concreting was started from one end block and continued towards the other. Bottom form vibrators were started when the bottom flange was full of concrete to about half its

height. Immersion vibrators were used continuously generally keeping one in the lower flange and web and one in the upper flange. Great care had to be taken to have proper vibration to ensure compaction of concrete in the corners of the lower flange and diaphragms. The top of the beam was finished simultaneously.

4.5.6. After the concreting was completed, it was necessary to check that no mortar had leaked into the cables and set there. This was done next day by hammering the cables at one end so that they moved out at the other. These were then hammered back in position.

4.5.7. It took 4 masons and 40 labourers about 8 hours to concrete one beam. Two mechanics were kept engaged in looking after the vibrators.

4.5.8. 6 to 12 cubes 4 inches by 4 inches by 4 inches were prepared at regular intervals for each day's concreting for testing in the laboratory.

4.5.9. As the concreting was completed in a certain length with the top surface properly finished, it was covered with wet gunny bags to prevent its drying.

4.6. Demoulding

4.6.1. The web form clamps were loosened about 4 hours after concreting. Sometimes it was not possible to take out the clamp bars and so these had to be cut.

4.6.2. Demoulding was started early next morning i.e. about 12 hours after completing of concreting and took two days with 5 carpenters and 10 beldars on the job.

4.6.3. The surface of beam as it was demoulded generally did not need any repairs except at the joints of the shuttering pieces where a line generally appeared, which had to be rubbed off. Honeycombed concrete was rare but where found all loose concrete in the neighbourhood of the patch was completely removed and replaced by new concrete. The best way to judge if the concrete is sound or loose is to hammer the concrete with a small hammer. A little experience will familiarize the engineer with the sound of the hammer on good concrete.

4.6.4. Curing of the beam was started as soon as the side forms were removed. This was done by covering the sides of the beam with gunny bags which were kept wet continuously.

4.7. Prestressing

4.7.1. Out of the ten cables in each beam one cable, usually No. 2, was prestressed on the third day of concreting. This was done to minimize shrinkage cracks in concrete. Three more cables No. 1, 3 & 4 were stressed after a week of concreting when the concrete had attained a strength of 4000 lb per sq. inch. The beam was then strong enough to support its own weight and was lifted up on jacks and bottom shuttering removed. The bottom of the beam could then be inspected and necessary repairs, if any, were carried out. Two more cables No. 5 and 6 were then stressed. The beam was now ready for launching. Cable No. 7 was prestressed in the span before the beam was allowed to carry the dead load of slab and cross beams. Cables No. 8 to 10 could only be stressed after the cross prestressing was completed and shuttering of deck slab and cross beams removed, because it was imperative that the beam should be supporting this dead load when last prestressing was carried out.

4.7.2. The equipment required for prestressing consisted of two Freyssinet jacks with their accessories, two hydraulic pumps, two scales marked in millimetres and two wooden struts $2\frac{1}{2}$ feet by $1\frac{1}{2}$ inches diameter. The staff required consisted of :

- | | | | |
|---|-----|-----|--------|
| 1. Engineer to supervise and record the observations | ... | ... | 1 No. |
| 2. Assistants to take the reading of the elongations at both ends | ... | ... | 2 Nos. |
| 3. Khalasis | ... | ... | 6 Nos. |
| They could do the fixing of jacks and later on do the pumping | | | |
| 4. Mechanic | ... | ... | 1 No. |

4.7.3. The prestressing consisted of the following operations which were similar at both ends:

(a) The 12 wires were first marked with chalk in the order in which they were placed in the cable. This is to prevent twisting of wires i.e. one wire riding over the other at a later stage. This is very important as it is difficult to find out the order of wires once they have been opened.

(b) The wires were then opened out and the core helix inside the cable was pulled out and cut at a suitable length (about 2 feet from the face of the core) and then pushed in. This

was done to prevent interference with the locking of the anchorage when the plug was pushed in.

(c) The male cone of the Freyssinet anchorage was then inserted. This cone is made of reinforced concrete and has 12 longitudinal grooves to take the high tensile wires. The wires were placed in these grooves and the plug was pushed inside the cone manually as far as it would go and then forced by light strokes through wooden struts, care being taken that the order of wires remained the same as marked with chalk earlier.

(d) The self-centring steel collar consisting of a split ring with an external disk was then placed round the cable in close contact with the anchorage cone. The bearing area of the cone should be smooth and have no raised spots. If there are any these should be chiselled off, because this is the area where ultimately the whole jack reaction comes.

(e) The prestressing jack was then lifted up and fitted in position, pushing the jack as near the steel collar as possible. The jack consisted of three parts. One was the head which had grooves in the periphery and formed the end of the moving piston. The second was the main cylinder in the rear, which had a wedge plate arrangement to fix the wires which reach it after passing through the grooves. The third part was another small piston working inside the grooved head. This small piston was used for locking purposes as detailed later.

(f) The wires were made to pass through the grooves in the head of the jack and then fixed in the rear by using wedge plates which were pushed in by hammering with a copper headed hammer. The fixing is shown in Fig. 10.

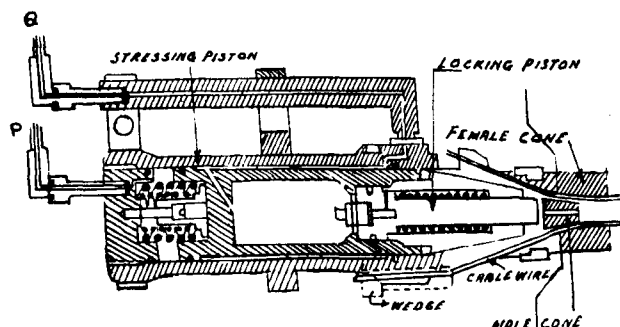


Fig. 10. Freyssinet jack

It was checked at this stage that the wires were quite clean and had no oil or grease on their surface.

(g) The jacks as above were fixed at both ends of the cable in the beam.

(h) For applying pressure to the jacks, hydraulic pump was used, which could apply pressure through two independent tubes. A line diagram of this pump is given in Fig. 11.

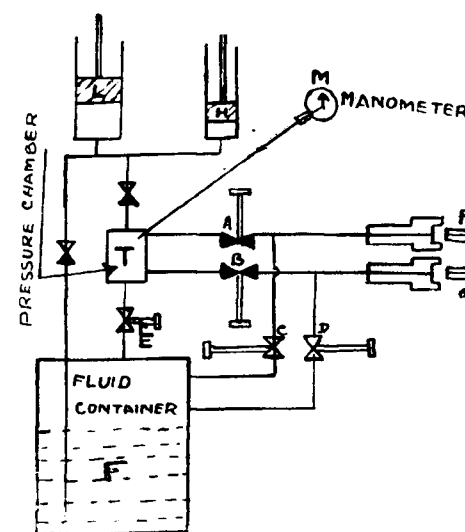


Fig. 11. Hydraulic pump

L and *H* are two pistons the former is a low pressure piston while the latter is a high pressure piston. *F* is the fluid chamber and *T* is the pressure chamber. *A* and *B* are the main valves and *C* and *D* are return valves for the two pressure pipes *P* and *Q*. *M* is the manometer which is connected to *T* and records the pressure. *E* is the valve which connects *T* and *F*. When it is desired to apply pressure to pipe *P* all valves are closed except *A* and *D*. Similarly when pressure is to be applied to pipe *Q* only valves *B* and *C* are kept open.

Pressure pipe *P* was connected to the tensioning chamber of the Freyssinet jack while pipe *Q* was connected to the locking chamber.

The main valve *A* of the pressure pipe *P* and the return valve *D* of the locking pipe *P* were first opened and all other valves were tightly closed. Some pressure was then applied, till the jack head was firmly set against the collar and the jacks became self-supporting. The wedges were then again hammered to ensure their tightness. It was very important to make sure that the wedges were not loose.

(i) The pressure was then worked up exactly 50 kg. per sq. cm. This removed any kinks in the wires, which were now all in low tension. This was the initial position from which observations were to be recorded.

(j) The scale was then placed with its zero end butting against the face of the female cone and a mark in red chalk or pencil was placed on one wire where the scale read a convenient figure say 500 mm. This formed the initial reading. When the cable was tensioned, the increase in the distance between the cone face and the red mark gave the elongation. Marks were also made on all the other wires close to the wedge grips to make sure that there was no movement relative to the grips. If the wedges were properly hammered in, there would be no such slippage.

(k) Pumping was then resumed and pressure raised to 100 kg. per sq. cm. and the distance between the cone face and the red mark measured. This reading less 500 gave the elongation of the wire. The elongations were observed at both ends totalled and recorded.

(l) The pressure was then raised successively to 200 kg. per sq. cm. and then 300 kg. per sq. cm., elongations observed and recorded.

(m) Finally the pressure was raised gradually to the value which was expected to give the required elongation which had been calculated beforehand and recorded in a table, a sample of which is shown in Table 5. If the required elongation was not obtained at the calculated pressure it meant that the friction between cable and sheath was high and the pressure was increased till the requisite elongation was obtained. The pressure was not to be raised beyond 400 kg. per sq. cm. However, in this bridge it was never found necessary to go upto that pressure. Observations recorded for one of the cables are shown in Table 5 A.

TABLE 5

Prestressing chart for beam F

CABLE No.				1	2	3	4	5	6	7	8	9	10		
CALCULATED VALUES	Length of Cable between female cones			ft	90'2"	91'0"	90'10"	90'6"	90'6"	90'4"	58'10"	66'6"	74'6"	82'2"	
	Length of Cable (Total)			ft	95'2"	96'0"	95'10"	95'6"	95'6"	95'4"	63'10"	71'8"	79'6"	87'2"	
	Maximum permissible pressure on piston of jack (Area 160.6 sq. cm.)			p. s. i.	6400	6400	6400	6400	6400	6400	6400	6400	6400	6400	
				kg/cm ²	450	450	450	450	450	450	450	450	450	450	450
	Calculated final pressure on piston of jack			p. s. i.	4750	4720	4620	4560	4630	4560	4920	4960	4990	5000	
			kg/cm ²	334	332	324	320	325	320	346	348	350	351		
Calculated final effective elongation			inches	4.78	4.82	4.77	4.72	4.74	4.71	3.23	3.66	4.09	4.48		
(from jack pressure 50 kg. per sq. cm.)			M	mm.	122	123	121	120	121	120	82	93	104	114	
OBSERVED VALUES	At Cable end A	Distance from mark to cone	At jack pressure 50 kg. per sq. cm.	A ₁		500	455	500	500	500	500	500	500	500	
			Just before locking	A ₂		579	542	576	555	573	572	542	552	555	545
			After locking	A ₃		576	535	570	551	570	566	535	544	550	538
		Final elongation (A ₃ - A ₁)		A		79	87	76	55	73	72	42	52	55	45
		Final pressure on piston of jack				370	340	360	350	350	350	400	400	400	400
	At Cable end B	Distance from mark to cone	At jack pressure 50 kg. per sq. cm.	B ₁		500	400	270	275	500	500	500	500	500	500
			Just before locking	B ₂		542	442	317	344	550	550	545	542	556	570
			After locking	B ₃		541	438	312	332	545	549	538	536	548	562
		Final elongation (B ₂ - B ₁)		B		42	42	47	69	50	50	45	42	56	70
		Final pressure on piston of jack				370	340	360	350	350	350	400	400	400	400
	Added final elongation A + B					121	129	123	124	123	122	87	94	111	115
	DATE					10/2	24/1	31/1	31/1	10/2	10/2	14/5	14/5	14/5	14/5

Note: *A* + *B* should be the same as *M*.

TABLE 5 A
Observations taken during prestressing

Date: 10-2-57		Beam No. F		Cable No. 5		
S.N.	Manometer reading kg per cm. ²	Observed reading in mm.		Elongation		
		Delhi end	Ghaziabad end	Delhi end	Ghaziabad end	Total
1.	50	500	500	—	—	—
2.	100	512	510	12	10	22
3.	200	533	525	33	25	58
4.	300	558	540	58	40	98
5.	350	573	550	73	50	123
6.	After locking	570	545	70	45	115

In the above case the required elongation as calculated was 121 mm. while the observed elongation came to 123 mm. which is satisfactory.

(n) When the final elongation had been obtained all the valves were closed and the main valve of locking pipe was opened gradually. This reduced the manometer pressure to zero. Pumping was then resumed, thus applying pressure to the locking piston in the jack. This piston abutted against the male cone and pushed the same inside the anchorage, thus locking the cable. Locking was carried out at a pressure of 300 kg. per sq. cm.

(o) Keeping the main valve of locking pipe open the return valve C of the pressure pipe was also opened gradually bringing the manometer pressure to zero. This removed the tensioning force from the wires of the cables.

(p) The elongation was now again measured and in all cases it was found to be less than that observed earlier before locking. These readings are entered at serial 6 in Table 5 A. This was due to two reasons :

- (i) Some slippage had taken place in the anchorage cone.
- (ii) The length of the wires between the anchorage and the wedges which had extended due to tension, had shortened as they were no more in tension. An allowance of a total loss in elongation of 12 mm. at

this stage had been made in the design but in the field it was found that the loss was always less. In the case illustrated in Table 5 A it was only 8 mm.

(q) To remove the jack now from the wires pumping was resumed, which made the inner piston move backward and strike the wedges out. The jack was then taken out along with the steel collar and fixed to another cable.

(r) The prestressed wires were tied with a small piece of wire. Photo 7 shows the prestressing in operation on cable No. 3. Cable No. 2 had been prestressed earlier.

4.8. Breaking of Wire

4.8.1. There was only one case in the whole prestressing of all the beams when a wire broke. This happened in cable No. 1 of beam No. D. The pressure had reached 350 kg. per sq. cm. but the elongation reached was only 94 mm. against 126 required. When the pressure was raised beyond 350 kg. per sq. cm. one wire broke. The jacks were thrown out at a distance of about 8 to 10 feet from the beam.

4.8.2. In this cable, then male cones at both ends were taken out and replaced with new ones. Remaining eleven wires were then tensioned to the required elongation which was obtained at 350 kg. per sq. cm. this time. It was decided to slightly overstress the remaining cables to compensate for the loss of one wire.

4.8.3. The breaking of the wire was either due to some kink in its length or some other defect which, however, could not be found out.

4.8.4. It may, however, be mentioned that this particular beam could not be used in the bridge as it toppled over when it was being lifted and was completely broken. A new beam was cast to replace it.

4.9. Lifting Beam from Casting Bed

4.9.1. After 4 cables had been stressed, the beam was lifted from the bottom shuttering.

4.9.2. For this the equipment required was as below :

- | | | |
|---|-----|--------|
| 1. Lifting jacks 30 tons capacity | ... | 4 Nos. |
| 2. Hydraulic pumps, same as those used for prestressing | ... | 2 " |
| 3. A few balli pieces | ... | ... |

4.9.3. Labour required was—

- | | | |
|----------------------------------|-----|--------|
| 1. Engineer to supervise the job | ... | 1 No. |
| 2. Khalasis | ... | 6 Nos. |
| 3. Carpenters | ... | 1 No. |

4.9.4. The removable part of the bottom shuttering was first removed. Two jacks on either side were then introduced below the beam. Concrete bed blocks had been laid earlier at places where the jacks were to be placed to distribute the load on the ground. It was checked that the top plates of the jacks were exactly horizontal.

4.9.5. Two jacks on one side were connected to the two pressure tubes of the hydraulic pump, and both were worked with the same pump simultaneously. This was done to ensure that both jacks went up or down together in case of any leakage in the pump, and the load was also distributed equally on both of them.

4.9.6. Pressure was applied to the jacks at one end of the beam, thereby lifting the beam. The other end was supported by means of balli struts on the sides to prevent toppling over of the beam.

4.9.7. When one end had been lifted to the required height, wooden sleepers were placed below the beam at this end. The jacks were then lowered and the beam was transferred on to wooden sleepers. The beam was also supported on the sides by using timber struts.

4.9.8. The other end was then lifted and placed similarly on wooden sleepers. Care was taken to see that the beam was horizontal in its final position.

4.10. Storing Beam in Yard

4.10.1. The beam was side rolled away from its casting bed to be stored till it was required for launching. This also cleared the casting bed for further making of beams.

4.10.2. For side rolling the beam from any position following equipment was required :

- | | | |
|---|-----|--------|
| (i) Lifting jacks capacity 30 tons each | ... | 4 Nos. |
| (ii) Hydraulic pumps, same as used for prestressing | ... | 2 Nos. |

- (iii) Mild steel rollers 4 inches diameter and 9 inches long ... 6 sets each consisting of 6 rollers connected together by two plates 3 inches by $\frac{3}{8}$ in. by 30 in. on either side.

(iv) **I-girders:**

2 Nos. each 6 feet long consisting of 2 plates 6 inches by $\frac{1}{2}$ inch as flanges welded to a web plate 5 inches by $\frac{1}{2}$ inch suitably stiffened by welding plates. At the ends of the top flange two pieces of angle iron $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{1}{4}$ inch and 6 inches long were welded.

- (v) Two plates 9 inches by $\frac{3}{4}$ inch for placing under rollers.

- (vi) Two chain pulley blocks 5 ton capacity.

- (vii) Wooden sleepers.

- (viii) Wooden ballies 4 inches diameter to be used as side struts.

4.10.3. Labour required was—

- | | |
|---------------------------|------------|
| (i) Engineer to supervise | ... 1 No. |
| (ii) Khalasis | ... 8 Nos. |
| (iii) Carpenter | ... 1 No. |

4.10.4. Side rolling platforms in R.C.C. had been erected in the casting yard for both ends of the beam.

4.10.5. One end of the beam was lifted and under it was placed first the I-girder and then the set of rollers. The $\frac{3}{4}$ in. plate was put under the roller. The beam was on either side supported by balli struts to prevent it from toppling on sides.

4.10.6. The other end of the beam was also similarly supported on the I-girder placed over rollers and supported by ballies.

4.10.7. Two chain pulley blocks were then tied round the beam, one at each end. The hooks of the chain pulley blocks were tied to some immovable objects e.g. trees or another beam in the casting yard. The beam was then pulled by the chain pulley

block gradually and simultaneously at both ends taking care to keep it straight.

4.10.8. When the beam had been rolled to the required position it was lifted again by means of jacks, and rollers and I-girders were removed, and the beam lowered on to wooden sleepers, one end at a time, as before, and supported by balli struts on either side.

4.10.9. It took about 2 hours to side roll a beam to a distance of 10 feet.

4.11. **Injection**

4.11.1. After a cable had been prestressed, it was to be injected with cement grout.

4.11.2. For this the lengths of the wires protruding outside the anchorage were cut leaving about 3 inches length which was fanned out.

4.11.3. For injection a pump working by a 2.5 H. P. electric motor was used. The pressure developed was 50 lb per sq. inch.

4.11.4. Water was first injected into the cable from one end to see that the passage was clear and to remove any dust or mortar inside the cable. The water which came out at the other end was allowed to flow for a few minutes.

4.11.5. Neat cement mortar made by mixing 2 parts cement with 1 part water and straining the suspension through a strainer was then injected. As cement grout was pumped in, it first pushed the water from the cable and then after some time the mortar would start coming out at the other end. At this stage a conical wooden plug was inserted at that end.

4.11.6. After the mortar had filled the whole cable tube it would start coming out backward from the end where it was being injected. The injector nozzle was then removed and this end also plugged.

4.11.7. The wooden plugs were left in position till the mortar inside the cable tube had set; say for a day or so.

4.11.8. When all the cables Nos. 1 to 6 had been injected and mortar had set, the plugs were taken out and the side plastered with 1 : 2 cement and coarses and mortar to have a smooth finish.

5. LAUNCHING

5.1. General

5.1.1. For launching of the beams which weighed 54 tons each it was planned to use the steel trusses of the existing bridge. From the casting yard the beams were taken on a trolley moving rail track to the span adjacent to the one in which it was going to be placed. From this span it was put on another trolley which moved over the girders of the existing bridge and took it into the span where it was to be lowered.

5.1.2. The distance between two steel trusses of the span was only 14 feet 6 inches and so beams could not be taken and placed in their final position. They were put together closely first and then moved out after the steel trusses had been dismantled.

5.2. Preparing Span to Receive the Beam

5.2.1. The decking of the old bridge consisted of steel troughs supported on cross beams and filled with kankar which was surface painted with bitumen. This was all removed except the cross beams which were allowed to remain to hold the trusses together.

5.2.2. Since the existing top of pier was at R. L. 678.13 and the bottom of bed blocks for the new bridge were to be at R. L. 674.36, it required dismantling of the pier top by 3.77 feet. This could not be done for the full pier top as that would have meant removal of the bearings of the steel trusses. Hence pier top was dismantled only in the central portion and bed block for the central beam was laid. After this block had been cured the pier top was ready to receive the beam.

5.3. Removing Beam from Yard

5.3.1. To move the beam from casting yard to the span adjacent to the one in which it was to be placed a track of 60 lb rails at 7 feet 6 inches centre to centre was laid along the centre of the roadway. Wooden sleepers 9 inches by 5 inches by 10 inches were used under the rails which were fixed to these by dog spikes.

5.3.2. Two trolleys were made to carry the beam on the track. Each trolley consisted of a steel channel frame with one axle and a pair of wheels, and was capable of carrying 30 tons load.

5.3.3. The beam was first side rolled from its position in the casting yard on to the centre of the rail track, and then it was transferred on to trolleys. For this the beam was lifted up by jacks and the trolleys introduced below it on the rails at either end, one end being lifted at a time. Wooden sleepers were placed on the trolleys below the beam to avoid cracking of the concrete at the corners of the lower flange. Balli struts were given at either side on each end to support the beam. Wooden wedges were placed between the wheels of trolley and rails on either side to prevent movement of the trolley during transfer of beam on to it.

5.3.4. To pull the trolley a crab winch of 5-ton capacity was used. This winch was fixed on proper foundation at one end of the bridge and steel rope one inch in diameter was fastened to the hooks provided at the end of the front trolley, the other end of the rope being fastened to the crab winch. As pull was applied by the crab winch, the trolleys moved forward carrying the beam.

5.3.5. The wheels of the trolleys had to be watched constantly as due to the rails not being straight at some places there was danger of derailment. At such places the trolley was moved slowly.

5.3.6. A beam could be rolled to a distance of about 300 feet in one day with the help of 16 khalasis and 2 mechanics. Photo 8 shows one beam on trolleys being rolled.

5.4. An Accident

5.4.1. An unfortunate accident occurred when a beam was being lifted in the yard from its supports to be side rolled to the rail trolley. The jacks had been placed under the beam ends and were being pumped when suddenly the beam toppled on its side and fell on the ground. It was broken and had to be discarded.

5.4.2. In this case two jacks which were placed under the end were being worked by two separate pumps. One of the jacks developed a leak and so the whole load was thrown on the other jack. The ground under this jack was not strong enough and settled on one side thus tilting the jack and the beam which toppled over. It was fortunate that the beam did not strike any neighbouring beam while going down.

5.4.3. For future, the following precautions were taken:

- (a) The beams were not stored closer together than their heights.

- (b) The two jacks placed under one end were worked by one hydraulic pump.
- (c) Sleepers were placed under the jack bases to spread the load over a wider area.

5.5. Putting Beam on Trolley moving on the Girders

5.5.1. There were two trollies moving on top flange of the steel girders, one of these carried the front end of the beam while the other carried the rear end. Each of these trollies consisted of a cross girder made up of two channels 15 inches by 4 inches arranged back to back with a clear gap of 4 inches in between. At each end of this girder and at right angles to it was fixed another member composed of two channels 9 inches by 3 inches size and 9 feet long. These members carried wheels at each end which moved on the girders. The wheels were 24 inches in diameter and 6 inches wide and were of cast iron with a mild steel sleeve. The axles of these wheels were 3 inches in diameter and two wheels were spaced at 6 feet. Over the cross girder there was another small trolley which moved on two rollers 4 inches diameter and 1 foot long and was composed of two channels 8 inches by 3 inches size resting on two verticals which in turn rested on the rollers. To carry the beam two bars 20 feet 6 inches long $2\frac{1}{2}$ inches diameter and square threaded in their whole length were placed vertically at 22 inches centres and passed in between the channels of the top trolley and the cross girder. Two pairs of nuts made out of 5 inches by 5 inches by 5 inches mild steel cubes were threaded on these bars, one pair to rest above the top trolley and the other pair above the cross girder. At the lower end of these bars two more nuts supported a platform composed of 2 channels 6 inches by 3 inches with a plate 18 inches by 10 inches by $\frac{3}{4}$ inch long welded at the top and two feet steel pieces 10 inches by 2 inches by $\frac{1}{2}$ inch at the bottom. The details of this trolley are shown in Plate III.

5.5.2. To place the beam on these trollies the following operations were carried out:

- (a) The V-shaped space between the two steel trusses was filled with vertical timber struts and an inverted channel was placed over it. This was done to provide a continuous moving surface for the top trollies.
- (b) Both the trollies were brought near the front end of the beam. One of these i. e. the rear one was pushed to the side and the front one was brought just near the front end of the beam and was so moved that

one vertical bar was on either side of the beam and passed through the two grooves left in end block for the purpose, and the bottom platform went under the bottom flange of the beam.

- (c) The nuts over the top trolley were tightened and the top trolley was lifted by putting a jack over the cross girder. This lifted the bars also and when the bottom platform touched and lifted the bottom flange of the beam a little bit the nuts on the bars above the cross girder were moved down to meet the cross girder. Now when the jack was lowered, the top trolley came down while the bars were left up being now supported by the nuts on the cross girder, and the beam end was transferred on to the bars. The nuts at the top of the top-trolley were also lowered down to meet the channels of the trolley.

5.5.3. The front trolley was now moved forward and in this stage the front end of the beam was on the girder trolley while the rear end was still on the rail trolley.

5.5.4. The top plate of the top member of the truss was only $\frac{3}{8}$ inch thick and was not found safe enough to take the load from the top trolley. Another steel plate $\frac{1}{2}$ inch thick and 12 inches wide was placed on the top of this member before the trolley was moved over it.

5.5.5. To roll the beam a long steel rope was tied to the front end of the beam by passing a $\frac{3}{4}$ in. mild steel rod through a hole in the end block and tying the rope to it to be pulled at the other end by means of a winch.

5.5.6. Before actual pulling was started the alignment of the wheels of both the trollies on the girders was checked. It was quite a problem to make the wheels of these trollies keep themselves in the centre. First, angle iron pieces were welded to these trollies on the inside to serve as guides in contact with the top beam of the girder, but these did not work as friction between these pieces and the steel girder was too much. Once the trolley had shifted to one side and the angle iron guides touched the steel girder, the beam had to be lifted off the trolley and the trolley adjusted which was quite a tedious job. An adjustment was therefore made in the wheels to make quick adjustments of these if possible.

5.5.7. A hole was cut oblong in shape in a mild steel plate 1 inch longer than the axle of the wheel. One such plate was welded to each channel of trollies on either side of the axle. Two

bolts were welded to each such plate projecting outside. To these bolts was fixed a plate having a central round hole for the axle of the wheel and two oblong holes for the bolts. The oblong holes allowed same play for the bolts as the former plate allowed for the axle of the wheel. These plates are shown in Fig. 12

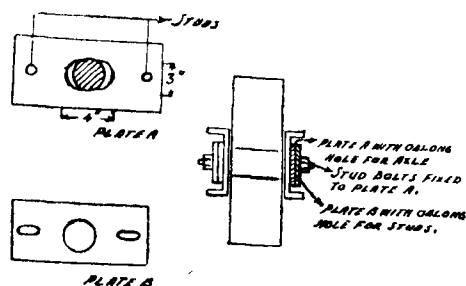


Fig. 12.

5.5.8. When it was found that the wheel had left the central line and had turned to one side, the trolley was stopped. The nuts on the bolts mentioned above were loosened and the wheel moved sideways by a crow-bar to bring it in the centre of the girder. The nuts were tightened again. This arrangement worked very satisfactorily and saved a lot of time.

5.5.9. When pulling the beam it was found that the channels of the cross girder bulged outwards. To check this $3/4$ in. diameter rods bent as shown in Fig. 13 were welded to the channels and wooden wedges inserted in between these rods and the channels. As an extra measure the two channels were tied together by chains also. In this way the beam could be moved easily.

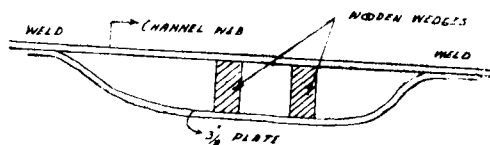


Fig. 13.

5.5.10. When the rear end of the beam reached the place where the front end was picked up on top trolley it was also picked up on the other trolley and then both trollies moved on steel girder.

5.5.11. The labour employed for rolling was—

1. Khalasis	...	20 Nos.
2. Mechanic	...	1 No.
3. Carpenters	...	2 Nos.
4. Beldars	...	4 Nos.

Launching of one beam could be completed in one day.

5.5.12. Photo 9 shows the rolling of beam on the top trolley.

5.6. Lowering Beam in Span

5.6.1. The beam was thus brought into the span into which it was to be placed and by side rolling of top trollies, it was centred in the span. A timber crib was erected on the bed block to a height just above the cross beams and the beam was lowered on this crib.

5.6.2. For lowering the beam a jack was placed on the cross girder under the top trolley. The nuts above the top trolley were moved up by about 9 inches. Now the jack was worked which lifted the top trolley (but not the beam); when the top trolley was lifted 9 inches it abutted against the top nuts and a slight lift beyond this transferred the weight of beam from the lower nut above the cross girder to the top nuts. The jack was stopped at this stage and lower nuts were moved up 9 inches when jack was lowered, the top trolley came down and along with it came down the beam which was supported on it by top nuts. When the top trolley came down 9 inches the beam weight was again transferred to lower nuts which by now had rested on the cross girder. This process was repeated for both ends of the beam till the beam was lowered to the desired level on the timber crib.

5.7. Placing Side Beams in Span

5.7.1. The side exterior beams were also brought similarly into span but before an exterior beam could be moved into the span for which it was meant it had to be side rolled on the top trollies, because the rail track being in the centre the beam was picked up on these trollies in the centre.

5.7.2. Vertical angle iron posts made up of two angle irons were welded to the cross girder of the top trolley at either end. To side roll a beam, a chain pulley block was tied to the top trolley at either end of the beam with the other end of the blocks

tied to the angle iron posts. The trollies were then pulled by the chain pulley blocks simultaneously on both trollies till the beam had reached the required position.

5.7.3. In practice it was found that the trolleys did not roll on the cross girder but the roller slid; however, that served the purpose all right.

5.7.4. These side beams were also lowered on timber cribs erected near the centre one, but the top of these cribs were kept about 2 feet higher than the centre one so that the web of side beams was clear of the flange of the centre beam.

5.8. Dismantling of Steel Truss

5.8.1. Once all the three beams had been placed in the span over timber cribs, the steel trusses could be dismantled in this span, but these trusses were also required for placing beams in the adjacent spans as the top trolley would rest on V-shaped gap for bringing beams in adjacent span. Thus the trusses in a span were actually dismantled when beams had been placed in the adjacent span also.

5.8.2. The first step in dismantling of trusses was the removal of the cross beams. As the removal of these beams would leave the trusses unbraced there was danger of their toppling over. Hence before the beams were removed the trusses were braced at the top by using inverted U-frames made of angle iron. The cross beam rivets were then cut away and the beams were lowered down into a boat stationed under the span to receive them.

5.8.3. The steel trusses were then lifted above their bearings by using jacks and were supported on steel rails which rested on top of the flanges of the exterior beams placed in the span and passed under the top beam of the truss. The support was provided at every panel point.

5.8.4. After supporting the truss in this way its dismantling was started with the bottom boom. Care was taken that dismantling was done equally in each truss to avoid the contingency of one side getting heavier and resulting in possible tilting of supporting rails which might then have resulted in sliding of one truss from its supports. The dismantled members were lowered into the boat as in the case of cross beams.

5.8.5. The bearings of the truss were also simultaneously removed. The pier top was dismantled and bed blocks for the exterior beams were poured.

5.8.6. When the bed blocks for exterior beams had been poured on the adjacent pier also, the beams could be side rolled into their final position.

5.9. Positioning Beam over the Piers in their Final Position

5.9.1. The central beam was first lowered on to the bearings, which had been set in the bed block. This was done by using jacks at both ends.

5.9.2. The exterior beams were also lowered as far as possible and then side rolled. For side rolling platforms were laid in concrete between the control and exterior bed blocks.

5.9.3. The process of side rolling was similar to the side rolling in the casting yard except that it was not possible to pull the beam by chain pulley blocks as there was no fixed object available to which the chain pulley block could be tied. To overcome this difficulty the lifting jacks were used in a lying position to push the beams.

5.9.4. The lifting jack was placed with its upper plate resting against the exterior beam to be moved and the bottom plate against the central beam which was already on its bearings. If the distance between the two was more than the length of the jack, wooden sleepers were introduced between the central beam and the jack.

5.9.5. As pressure was applied to the jack by the pump the beam moved out. The operations were carried out simultaneously at both ends so that the beam remained always parallel to the central beam. When the beam had been side rolled to its final position, it was lifted up with the jack and lowered into position over the bearing.

6. CROSS BEAMS & DECK SLAB

6.1. Shuttering

6.1.1. The cross beams and deck slab were cast-in-situ. The shuttering for cross beams was suspended from 9 in. by $4\frac{1}{2}$ in. sleepers supported on the top flanges of the main beam. For the deck slab sleepers were supported on bottom flange of the beam. On these sleepers verticals were fixed. On top of these verticals again transverse members were fixed and wedged between webs of beams. On top of these transverse members longitudinals were run. On top of the longitudinals rectangular steel panels were placed to form the shuttering for the slab. No

side shuttering was required for deck slab as the top flanges of the main beams themselves acted as such. Details of the shuttering for cross beams and deck slab are shown in Plate IV.

6.1.2. Prestressing cables were placed in position giving them the desired eccentricity in case of deck slab. The cables passed through the holes left in the main bearing while in the portion to be cast-in-situ they were put inside cable tubes. Mild steel reinforcement was also fixed in position.

6.2. Concreting

6.2.1. The mix used for the cross beams and deck slab was the same as the main beams. The cross beams were first concreted and then the deck slab.

6.2.2. The total quantity of concrete in these members in one span was 890 cu. ft.

6.3. Prestressing

6.3.1. The anchorage cones for the 2 prestressing cables in each cross beam had been fixed in the exterior main beams while concreting them. For the cables in deck slab space had been left in the exterior beam flange for the anchorage cones. In the slab the anchorage cone was fixed only at one end. At the other end 2 bars $1\frac{1}{2}$ in. diameter and 8 in. long were used round which the high tensile wire was looped. Thus 12 wire cables were formed by using 6 wires which after passing through the conduit were again passed through it after turning round the bars as shown in Fig. 14.

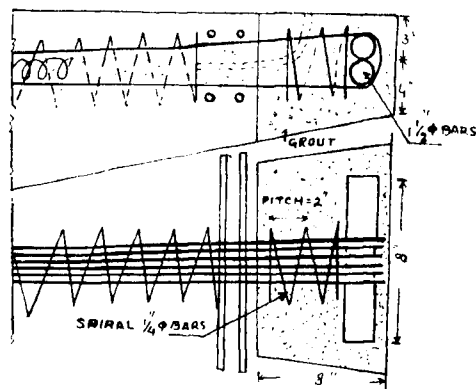


Fig. 14. Cable in deck slab

6.3.2. Hoop reinforcement was provided near the bars where wires were turned round as the stresses here would be high. A small steel tube connected to the cable was left in the concrete to allow ventilation when injection was done.

6.3.3. One central cable of each cross beam was prestressed 3 days after the concreting. With this cross beams and deck slab had enough prestress to take up their own dead load and so the shuttering was removed. The remaining cables of the beam and deck were stressed after 7 days of concreting.

6.3.4. In the cross beam cables, male cone was inserted at both ends but the cable was stressed from one end only, generally downstream side. There was some slippage in the upstream side anchorage which was measured by putting marks on the wires and measuring the distance from face of cone to these marks before and after prestressing. This value was deducted from the elongation observed on the downstream side to arrive at the actual elongation. If in any cases the required elongation was not obtained from one side even when maximum permissible force was applied, which might be due to excessive friction in the cable due to some cause, this cable was stressed from upstream side also and the balance elongation was obtained on that side. Plate V shows how observations were recorded for deck slab and X-beams.

6.3.5. The jacks for prestressing these cables had to be supported from a derrick of wooden ballies erected on the edge of the top flange of the main beams. To keep them from falling into the river, the wedges of the jack had to be anchored to the body of the jack by means of a chain passing through hooks welded to the wedges.

6.4. Injection

6.4.1. The cables for deck slab and cross beams were injected in a way similar to the injection of main beam cables. The tubes left for ventilation on the upstream side of the deck slab cables were cut and plastered after injection. The spaces for the anchorage were also filled with concrete and given a smooth finish.

7. CURB, RAILING & WEARING COAT

7.1. Curb & Railing

7.1.1. The shuttering for the curbs was supported with the reinforcement which had been fixed in the main beam. The curbs projected 6 inches outside the deck slab.

7.1.2. The precast vertical posts of the railing were fixed in the curb while concreting. They were aligned in a straight line by means of a theodolite.

7.1.3. The members of the railing which had also been precast in advance, were fixed to the vertical posts after the concrete of the curb had set.

7.1.4. The mix used for the curbs and railing was 1 : 2 : 4.

7.2. Wearing Coat

7.2.1. The wearing coat was also laid in 1 : 2 : 4 mix.

7.2.2. A layer of water proof paper was spread over the slab before laying the wearing coat to ensure the separation of the two.

7.2.3. The steel angle irons for expansion joints were also fixed in position, as they were anchored in the wearing coat.

7.2.4. Two expansion joints were provided longitudinally at either end between the wearing coat and the curbs. In these joints pre-fabricated bituminous expansion jointing material was filled.

7.2.5. There was one construction joint at 12 inches from each expansion joint at either end of each span. The portion which was provided as wearing coat near the expansion joint was poured first to anchor the angles. One transverse expansion joint was provided in the centre of each span, thus making a panel of the wearing coat equal to about 45 feet by 24 feet. It has, however, been found that these panels were rather too big and shorter panels with more expansion joints should have been provided.

7.2.6. Reinforcement in the wearing coat consisted of a mesh of $1\frac{1}{4}$ in. diameter bars 1 foot centre to centre both ways placed in the middle of the slab.

7.2.7. The thickness of the wearing coat was kept as $3\frac{1}{2}$ inches in the centre and $2\frac{1}{2}$ inches at the ends, the top surface being a parabola. To check the camber a wooden truss had to be made with its bottom having the desired camber, but it proved to be rather unwieldy. A central longitudinal expansion joint, if provided, would have made it easier to control the camber.

8. TESTS

8.1. The cubes cast with concreting of various parts of the bridge were tested in the laboratories of Hindustan Housing Factory, Delhi and P.W.D. Research Station, Lucknow. The average strength of cubes was found to be as follows :

4 days strength	4250 lb per sq. in.
7 days strength	5450 lb per sq. in.
28 days strength	6575 lb per sq. in.

8.2. Load Test

8.2.1. A full load test was carried out in span No. 2 to test one of the beams. A load equal to the class AA track load which gives the maximum bending moment for beams was applied in such a position as to create maximum bending moment in the downstream exterior beam.

8.2.2. The class AA load consists of a tank of 70 tons. The tank has 2 tracks 12 feet long and 2 feet 9 inches wide. The minimum clearance between the roadway face of the curb and the outer edge of the track, as per I.R.C. Bridge Code, is 4 feet for a multiple lane bridge (width more than 18 feet). For maximum bending moment the tank should be in the centre of the spans. All these conditions show that for maximum bending moment in the exterior beam the load should be placed as shown in Fig. 3. The impact factor is 10 per cent, giving the total load to be put in as 77 tons.

8.2.3. The loading in the span was done by means of sand bags loaded over a platform made of sleepers. The actual arrangement of loading is shown in Fig. 15.

8.2.4. Two parallel platforms 2 feet 9 inches wide and 12 feet long, 4 feet apart, were first made with two layers of wooden sleepers, the outer edge of the platform nearest to the edge being 4 feet from the face of the curb. These formed the two tracks of the tank. Across these platforms and on them were placed 6 numbers steel cross girders dismantled from the old bridge. A platform 14 feet 6 inches by 12 feet was made of one layer of

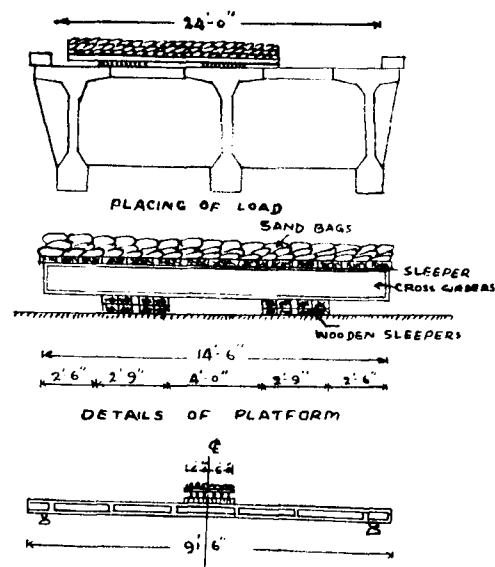


Fig. 15. Load test

wooden sleepers, over these girders. The platform was loaded with sand bags. Each bag was filled with sand to weigh exactly 1 cwt. The weight of the lower layer of sleepers, cross girders and upper platform of sleepers came to 5.9 tons. A load of 71.1 tons was put in by means of 1422 bags.

8.2.5. Observations were taken by means of a levelling instrument for the deflection of all the three beams. It is a fact that a levelling instrument does not have so much sensitivity as required in such cases, but as the depth of water in the span was considerable, no arrangement was possible to magnify the deflection. Two reference pillars were erected in the two adjacent spans and the points on all the beams at $1/4$ th span, $1/3$ rd span and centre were recorded. It was found that the deflection in the centre of the downstream beam was 0.07 feet while the deflections in the centre of the central beam and the upstream beam were 0.05 feet and 0.03 feet respectively as given in Table 6.

TABLE 6

Deflection

Sl. No.	Loading in tons	Downstream Beam					Central Beam					Upstream Beam				
		$1/4 L$	$1/3 L$	$1/2 L$	$2/3 L$	$3/4 L$	$1/4 L$	$1/3 L$	$1/2 L$	$2/3 L$	$3/4 L$	$1/4 L$	$1/3 L$	$1/2 L$	$2/3 L$	$3/4 L$
1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	10	0	0	.02	.01	.01	0	0	.01	0	0	0	0	.01	0	0
3	20	.01	.01	.03	.02	.02	0	0	.02	0	.01	0	0	.02	.01	.01
4	40	.01	.01	.03	.02	.02	0	0	.02	.01	.02	0	0	.02	.01	.01
5	50	.01	.03	.04	.03	.02	.01	.02	.02	.01	.02	.01	.01	.02	.01	.01
6	60	.03	.04	.05	.04	.03	.02	.03	.04	.02	.02	.02	.02	.02	.02	.02
7	70	.03	.05	.06	.04	.03	.03	.04	.04	.02	.02	.03	.03	.02	.02	.02
8	77	.03	.05	.07	.05	.04	.03	.04	.05	.03	.03	.03	.03	.03	.03	.02

8.2.6. For the position of the load as above assuming the cross beams to be indefinitely rigid in comparison to the main beam, resulting in linear distribution of load between the three beams as per para. 2.5.6 the three beams carried the following portions of the load :

Left beam	...	0.488
Centre beam	...	0.333
Right beam	...	0.178

The ratio of the loads was 2.7 : 1.8 : 1.

The ratio of observed deflections was .07 : .05 : .03 or 2.3 : 1.7 : 1. These were very nearly the same and so the assumption of load distribution was on right lines.

8.2.7. In calculating the theoretical value of the deflection of the beam, a very uncertain factor is the modulus of elasticity of concrete. Another factor which makes exact calculations difficult is the variation in the moment of inertia of the beam, from section to section and the difference in the number of cables and their position at each section. Therefore, the value of E was calculated backward from the observed deflection and it worked out to 2.85×10^6 lb per sq. inch as shown below:

8.2.8. For a beam loaded in the centre by a load P , uniformly distributed through a distance ' a ' (less than the span of the beam l), the deflection at the centre is given by

$$\delta = \frac{P}{384EI} (8l^3 - 4a^2l + a^3)$$

where

P = Load applied = 0.488×77 = 37.6 tons.

l = Length of the beam = 88.33 feet.

a = Distance through which load acts = 12 feet.

I = Moment of inertia of the section = 862,000 in⁴.

δ = Deflection = 0.07×12 = 0.84 inch.

E = Modulus of elasticity of the material of the

beam.

Substituting the above value in the equation—

$$0.84 = \frac{37.6 \times 2240}{384 \times E \times 862000} \left[8(88.33)^3 - 4 \times 144 \times 88.33 + (12)^3 \right] \times (12)^3$$

$$\text{or } E = \frac{37.6 \times 2240 \times 5549728 \times 1728}{384 \times 862000 \times 0.84}$$

$$= 2.85 \times 10^6 \text{ lb per sq. inch.}$$

8.2.9. Since the value of the moment of inertia of the beam at other sections would be less than the value taken in the above calculations the value of E would be a little more than this value. Guyon in his book on Prestressed Concrete recommends a value of $E = 2.1 \times 10^6$ lb per sq. inch for deformations due to permanently applied loads and this is to be used when it is required to calculate the homogeneous section properties.

9. DIVERSION

9.1. Since the bridge carried a very heavy traffic it was essential to construct a diversion which could serve efficiently

without much inconvenience to the traffic. The depth of water in the river remains considerable during dry weather due to the weir downstream remaining closed. A wooden pile bridge 312 feet long was constructed about 350 feet downstream of the bridge for the diversion. Wooden balli piles with a minimum diameter of 6 inches were used.

9.2. The deck consisted of salwood sleepers carried over salwood beams. Metalled approaches with surface painting were provided to the pile bridge.

9.3. Since the pile bridge was only 12 feet wide, only one way traffic was allowed on it. Barriers were erected on both sides and traffic was controlled by P. W. D. staff. Adequate lighting arrangements were made on the pile bridge and its approaches.

10. COST DETAILS

The cost of various items worked out to be as follows:

(i)	Total cost of the superstructure including bearings	...	Rs	3,30,000
(ii)	Cost of the superstructure per running foot	...	Rs	714
(iii)	Cost of the superstructure per sq. ft. of travelling surface	...	Rs	30
(iv)	Cost of dismantling the old steel girder bridge	...	Rs	15,000
(v)	Cost of temporary diversion including maintenance but excluding cost of salvaged materials	...	Rs	70,000

11. VISITORS

The bridge was visited during its construction by a number of eminent persons including engineers. One of the prominent engineers to visit the bridge was Monsieur Y. Guyon, Technical Manager of the Societe Technique Pour L' Utilization De La Precontainte and an authority on Prestressed Concrete. His visit to the site offered an opportunity to local engineers to discuss with him problems on prestressed concrete.

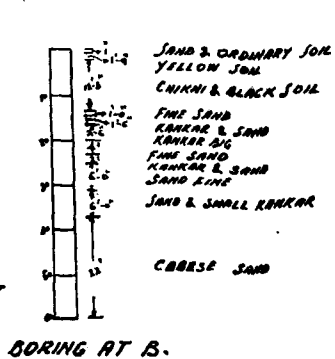
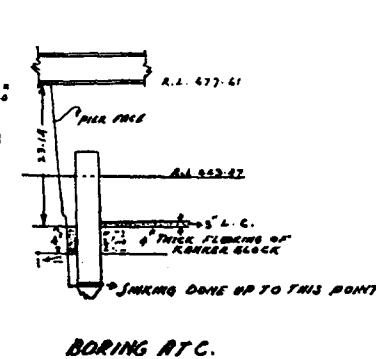
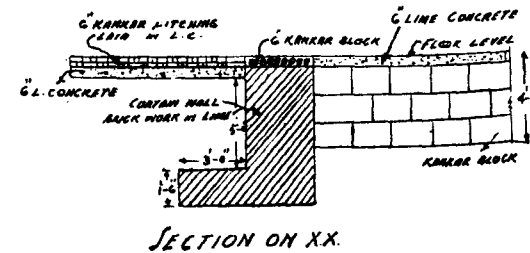
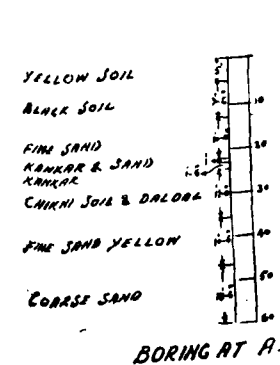
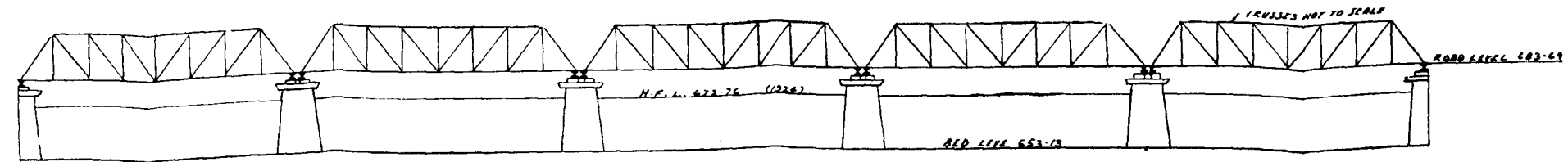
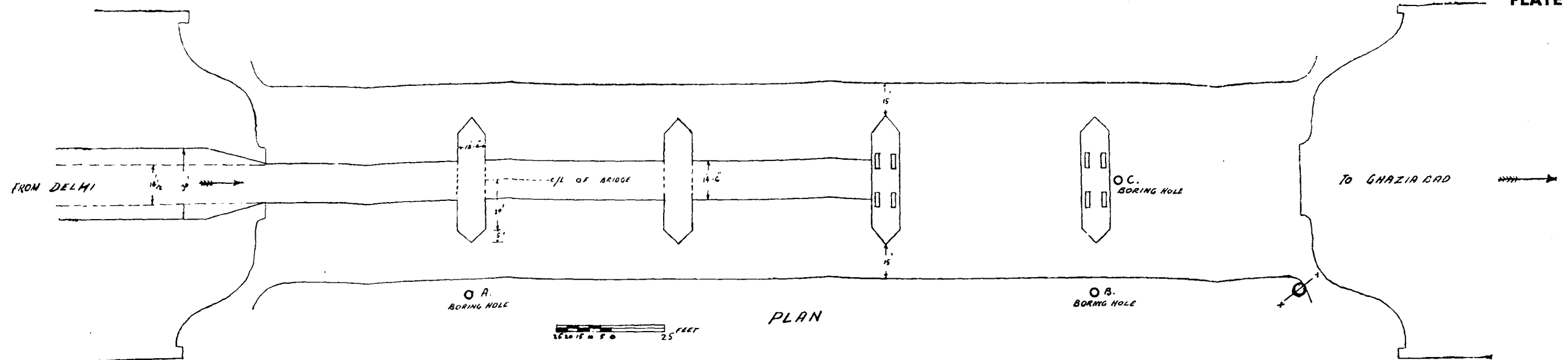
ACKNOWLEDGMENTS

The Paper has been written with the kind permission of the Chief Engineer, Uttar Pradesh Public Works Department.

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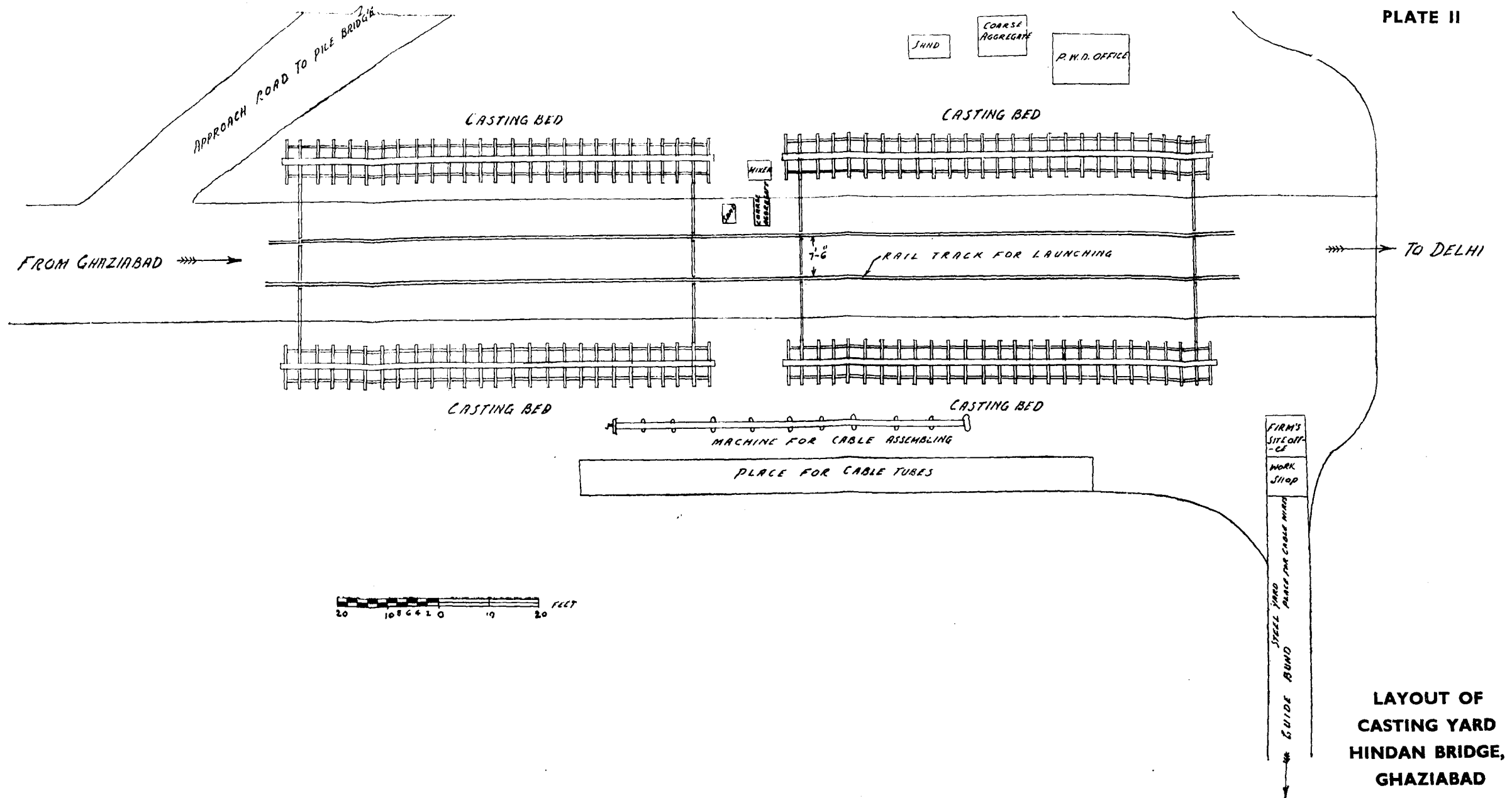
REFERENCES

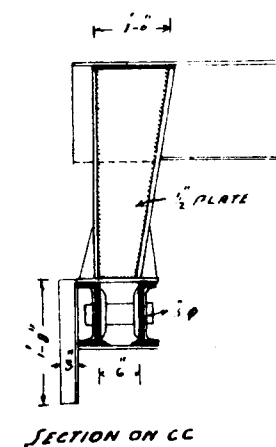
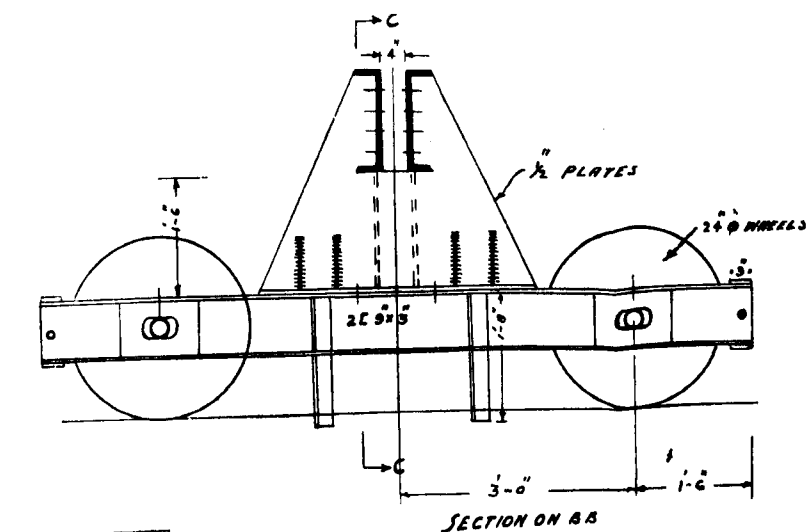
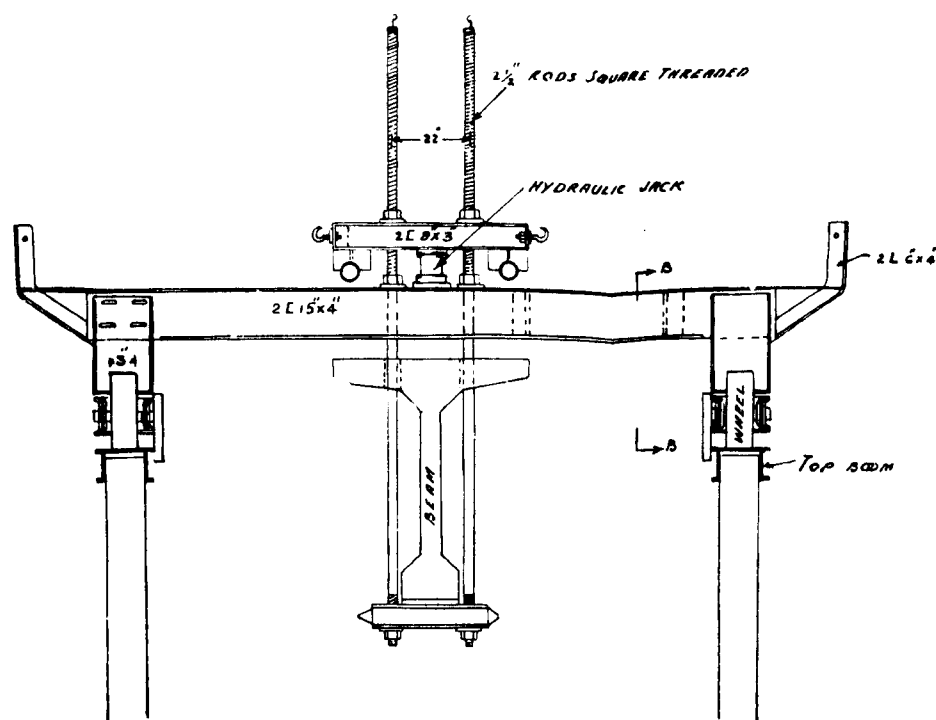
1. "Concrete Plain & Reinforced" Part II—Taylor Thomson & Smulski.
 2. Prestressed Concrete—Y. Guyon, Contractors Record Limited, London, 1957.
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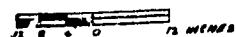
OLD BRIDGE WITH
FOUNDATION
DETAILS—HINDAN
BRIDGE, GHAZIABAD

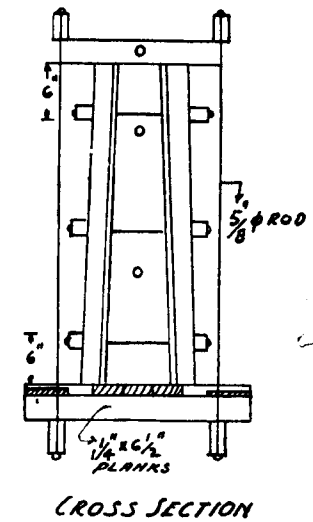
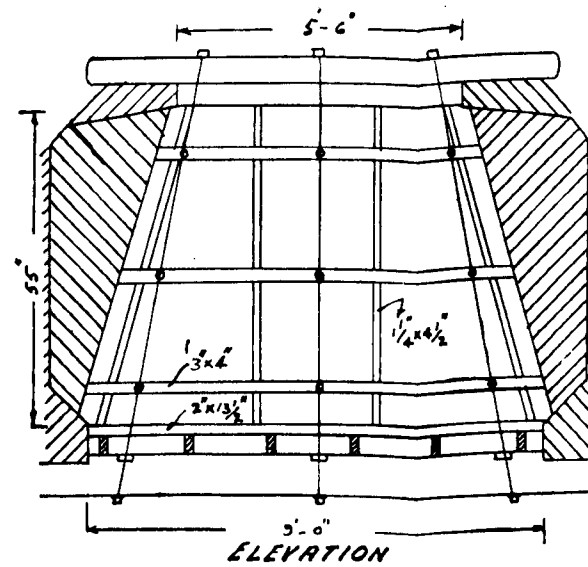
PLATE II



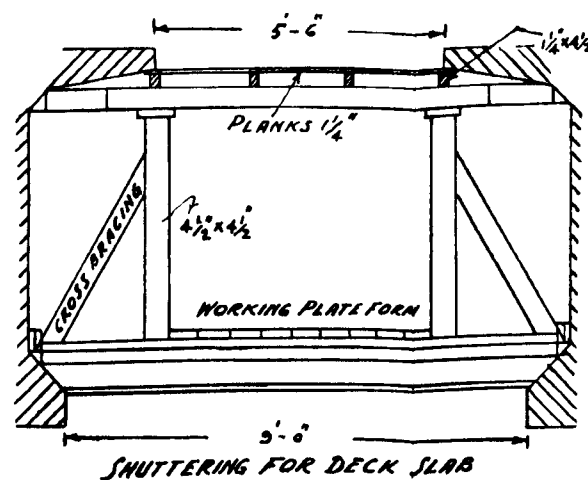
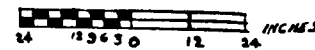


**TROLLEY ROLLING
ON TOP OF EXISTING
GIRDERS—HINDAN
BRIDGE, GHAZIABAD**





SHUTTERING FOR CROSS BEAM



DETAILS OF SHUTTERING
FOR X BEAMS & DECK
HINDAN BRIDGE,
GHAZIABAD

PLATE V

Cross-Girders Cable 1-6 are the upper cables 7-12 the lower.

Cable No.		1	2	3	4	5	6	7	8	9	10	11	12
Length of Cable (between female Cones)	- ft	23'-6"	23'-6"	23'-6"	23'-6"	23'-6"	23'-6"	22'-2"	22'-2"	22'-2"	22'-2"	22'-2"	22'-2"
Length of Cable (Total)	ft	28'-6"	28'-6"	28'-6"	28'-6"	28'-6"	28'-6"	27'-2"	27'-2"	27'-2"	27'-2"	27'-2"	27'-2"
Maximum Permissible Pressure on Piston of Jack $A = 160.6 \text{ cm}^2$	Kg/cm ²	450	450	450	450	450	450	450	450	450	450	450	450
Calculated final Pressure on Piston of Jack	Kg/cm ²	500	500	500	500	500	500	500	500	500	500	500	500
Calculated final effective elongation (from Jack Pressure 50 Kg/cm ² ~ 710 lbs)	M	1.77	1.77	1.77	1.77	1.77	1.77	1.69	1.69	1.69	1.69	1.69	1.69
At Jack Pressure 50 Kg/cm ²	A ₁	500	500	500	500	500	500	500	500	500	500	500	500
Just before locking	A ₂	545	546	545	530	542	542	549	546	542	545	548	540
After locking	A ₃	540	541	543	525	538	536	543	538	536	540	542	534
Final elongation $A_2 - A_1$	A	45	46	45	30	42	42	49	46	42	45	48	40
Final Pressure on Piston of Jack	-	400	390	400	380	380	400	400	400	400	400	400	400
At Jack Pressure 50 Kg/cm ²	B ₁				500	500	500						
Just before locking	B ₂				522	522	509						
After locking	B ₃				517	516	507						
Final elongation $B_2 - B_1$	B	0	0	0	17	16	7	-10	-2	-5	-5	-5	0
Final Pressure on Piston of Jack	-				350	350	400						
Added final effective elongation A+B (Must be the same as M above)	A+B	45	46	45	47	58	49	39	44	37	40	43	40

BRIDGE DECK ON SPAN NO. 1

Cable No.		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Length of Cable (between female Cones)	ft	24'-8"																				
Length of Cable (Total)	ft	29'-8"																				
Maximum Permissible Pressure on Piston of Jack $A = 160.6 \text{ cm}^2$	Kg/cm ²	450																				
Calculated final Pressure on Piston of Jack	Kg/cm ²	500																				
Calculated final effective elongation (from Jack Pressure 50 Kg/cm ² ~ 710 lbs)	M	1.81																				
At Jack Pressure 50 Kg/cm ²	A ₁	700	600	600	600	600	600	600	500	600	600	500	700	700	700	700	700	700	700	700	700	500
Just before locking	A ₂	748	647	646	645	646	646	646	545	647	647	545	748	747	746	746	745	748	738	747	745	546
After locking	A ₃	745	639	638	638	638	638	637	535	640	640	536	740	740	740	740	740	742	734	741	740	539
Final elongation $A_2 - A_1$	A	48	47	46	45	46	46	46	45	47	47	45	48	47	46	46	45	48	38	47	45	46
Final Pressure on Piston of Jack	-	400	380	400	400	400	400	390	390	400	390	380	400	400	400	380	400	400	400	380	380	390

**“EXPERIMENTAL SOIL STABILIZED ROAD
CONSTRUCTION IN BLACK COTTON SOIL AREA ”**

By

J. F. MISTRY*, B.E. (CIVIL) HONS.

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SYNOPSIS

The Paper deals with experimental work of soil stabilized base course on Dabhoi-Bodeli road in Baroda District of Bombay State. This road passes through a black cotton soil area.

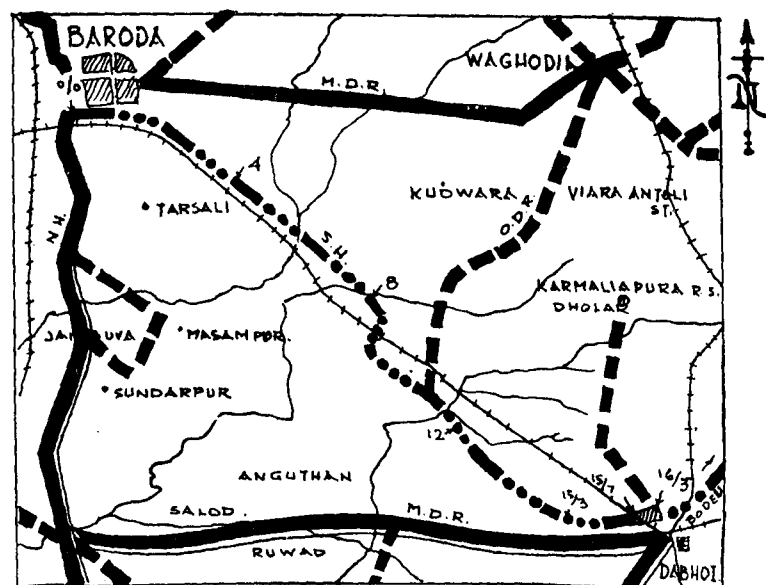
Experimental work done in the laboratory with local soils and their combinations is described. The design of the pavement arrived at and the method of construction are also described.

From cost data given it is seen that stabilized soil base course effects a saving of 42.48 per cent per 100 sq. ft. over $4\frac{1}{2}$ in. thick brick soling.

1. SHORT HISTORY OF THE PROJECT

1.1. The junction of the Dabhoi-Bodeli road and Baroda-Dabhoi road is in mile 15/3 (Index Map). The Baroda-Dabhoi-Bodeli road is going to be classified as a State Highway. The whole length of this road passes through black cotton soil area. The road from Baroda to Dabhoi is being constructed with $4\frac{1}{2}$ in. brick soling plus $4\frac{1}{2}$ in. metal ($1\frac{1}{2}$ to 2 in. size) and $\frac{3}{4}$ in. carpet

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NOTE:-

EXPERIMENTAL LENGTH OF SOIL STABILISED

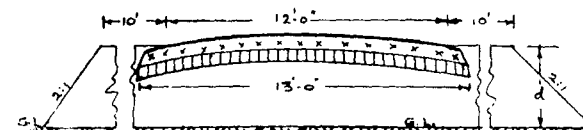
DABHOI - BODELI ROAD (MILE NO 15/7 TO 16/3) SHOWN THUS

MILES 4 2 0 4 MILES

Index Map.

over it as per Fig. 1. The total thickness of $9\frac{3}{4}$ in. above the poor subgrade of black cotton soil has proved very unsuccessful in miles 0/0 to 2/0 and 11/6 to 15/3, even though asphalt carpet was already laid. The subgrade has yielded unevenly and there are a number of pot holes and ruts and corrugations in the above lengths of the road. This is due to the peculiar property of black cotton soil which is highly clayey. It becomes hard when dry and very soft when wet. It loses its strength considerably, some soil not being capable of taking more than 1,000 lb per sq. ft. of load. It expands and contracts greatly when wet and dry to the extent of 20 to 30 per cent of its original volume. In the dry weather, the whole area shrinks and splits up into large cracks. Sometimes 3 to 6 in. wide cracks at the surface and extending to 10 in. or so below the ground are formed where the soil is thick.

As its bearing power becomes very poor when wet, it is seen from the pressure-bulbs developed under a concentrated load



CONVENTIONAL DESIGN OF ROAD-PAVEMENT
ADOPTED BY THE DIVISION FOR STATE-HIGHWAY.

	ASPHALT CARPET	$\frac{3}{4}$ INCH.
	METALLING	$4\frac{1}{2}$ INCH.
	BRICK SOLING	$4\frac{1}{2}$ INCH.
	FILLING DONE IN LAYERS, ROLLED & COMPACTED.	d INCHES.
	NATURAL SUBGRADE	—

Fig. 1.

that the pressure bulb of the least intensity should reach the poor subgrade of black cotton soil. Thus we come to the conclusion that the wheel load must be at a certain distance above the subgrade so that the pressure transferred to the subgrade is the minimum.

1.2. The construction of the Dabhoi-Bodeli road (estimated to cost Rs 16,60,500) was proposed to be done in the conventional manner as per Fig. 1, i.e., using brick soling. But in practice it was observed from mile 0/0 to 2/1, where such type of design was used, that the road had been badly deteriorated in one season. As such, it was decided that further work on this road in the usual conventional way would be of no use, but some rational design looking to the poor subgrade met with had to be worked out. As an experimental measure, it was decided to take up mile 15/7 to 16/3. Construction in this portion was then proposed to be done in layers of stabilized soil, making use of the knowledge of soil proportions and California Bearing Ratio design method utilising local soil, sandy soil and sand available from nearby areas,

1.3. The road carries mixed traffic. A traffic census survey conducted on the Baroda-Dabhoi road on 28-5-1956 (24 hours) gave the following results :

Private cars	... 18 No.	22 Tons
Passenger buses	... 27 „	230 „
Trucks	... 33 „	213 „
Bullock carts (iron-tyred)	... 151 „	150 „
		615 tons
Average per hour	...	25.5 tons

The future intensity of traffic on this road is likely to increase as a result of improvement of the Baroda-Dabhoi-Bodeli State Highway which is in hand. Development of minor and other connecting feeder roads will also increase it further.

2. LABORATORY INVESTIGATIONS

2.1. Analysis of Soil Samples

Stabilization implies increasing the stability of material under given set of moisture conditions by using correctly proportioned mixture of coarse and fine aggregate like gravel, silt, clay, fully compacted to give a good stable base or surfacing. The above method of construction is termed as Mechanical Stabilization. The principal properties affecting the stability of compacted granular materials are the internal friction and cohesion, the former being the characteristics of the coarser soil particles like gravels, sands, etc., and the latter of the clay fractions.

Most aggregates with crushing strengths above 12,000 lb per sq. in. or aggregate crushing value below 40 are believed to be sufficiently strong for the purpose. They should have sufficient mechanical strength to retain approximately the same size distribution during compaction and later use.

It was proposed to take up the experiment of soil stabilization for base course on Dabhoi-Bodeli road in mile 15/7 to 16/3 by mechanical stabilization, as the other methods by the use of cement, bitumen or other binders were costly as the soil here is very clayey and hence the proportions of bitumen, cement, etc., required would be very high.

Samples of natural sand from the nullah bed, sandy soil from two fields (Survey No. 274) near the nullah culvert at Vega village and five samples of the subgrade taken at different spots in the above mentioned 4-furlong length, were brought to the Research Laboratory, Baroda, for conducting various tests and to see whether it was possible to utilise them when suitably proportioned to give a dense stable base course.

To utilize the inherent properties of soil received and to provide cohesion and friction for mechanical stabilization, it was necessary to carry out the particle size analysis and the B.S. plasticity tests and, if necessary, 'Compaction' and 'C.B.R.' tests on mixture to see to what extent these properties could be made use of.

The results of tests and analyses of the three types of soil selected are given in Tables 1 to 3.

2.2. Proportioning the Materials

Natural materials are found generally deficient in one or more of the particle size fractions required and a mechanically stable material can be prepared only by mixing two or more of them in the correct proportions. Numerous methods have been prescribed for this purpose but the quick and simple method described by Rothfuchs¹, in which two or more materials of known sizes are mixed to produce a specified size distribution, has been found most useful. The essential stages are :

(1) Specified cumulative particle size distribution curve is plotted, using linear ordinates for the percentages passing but choosing a scale of sieve size such that the specified particle size distribution (Table 4, Col. 3) plots as a straight line and marking on it the sizes corresponding to various percentages passing.

(2) The particle size distribution curves of other samples to be mixed are plotted on this scale which may not be straight lines. Curve AXYZP is for sandy soil and curve AQRSTUV is for pure sand, Fig. 2.

(3) Using a transparent straight edge, a straight line is drawn for every curve such that the areas enclosed between it and the curve are minimum and are balanced about the straight line, i.e., straight lines AB and CD.

(4) The opposite ends of these straight lines, i.e., end B of AB and end C of CD, are joined together to intersect the straight line AV (the line showing the specified size distribution for $\frac{2}{3}$ in. maximum base) in E. The proportion for mixing can be read

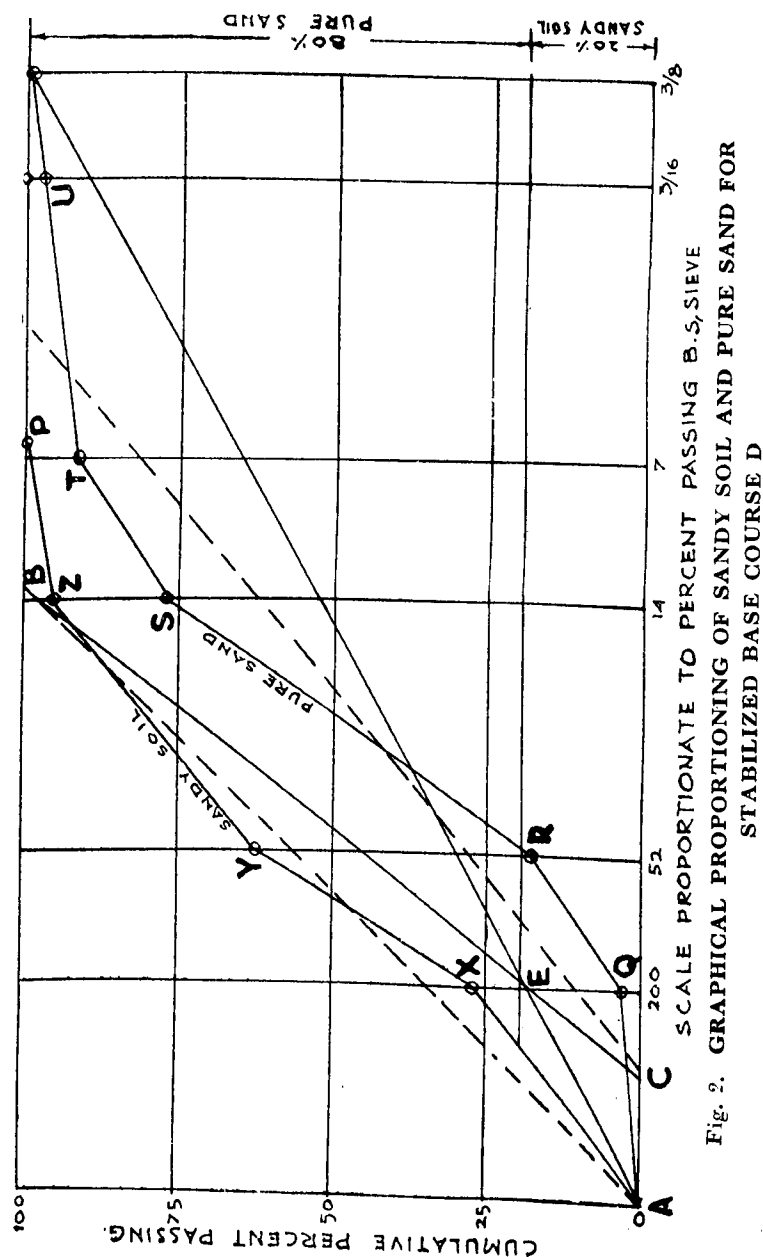


Fig. 2. GRAPHICAL PROPORTIONING OF SANDY SOIL AND PURE SAND FOR STABILIZED BASE COURSE D

off from the point E which lies on 20 per cent of the ordinate and divides the mix in 20 : 80 proportion.

The pure sand available from the nullah bed near Vega village and the sandy soil locally available from Survey No. 274 were tried in various proportions and the resultant particle-size distribution is given in Table 4. According to Fig. 2, the proportion of mix of sandy soil to pure sand was 1 : 4. This mixture gave a C.B.R. value higher than 20 but, on account of limited quantity of pure sand available from the running nullah near the site, the actual proportion was changed to 1 : 2 for reasons of economy.

2.3. Pavement Design for a Base Course based on C.B.R. Value

There are two main difficulties in devising a sound theoretical method of pavement design : (1) No existing analysis of the distribution of stress and strain through the layers of the pavement and subgrade has yet been proved sufficiently accurately and also it is not known whether it is necessary to take into account non-elastic behaviour of the materials ; (2) No methods have yet been devised to measure the strength properties of the materials for use in order to take account of such factors as load repetition. Therefore, an entirely empirical approach is sought for selecting an arbitrary strength test which is considered to stress the subgrade in a manner similar to an actual wheel load. The thickness of the pavement is then determined on the basis of experience of thickness required in the past over a subgrade of similar strength.

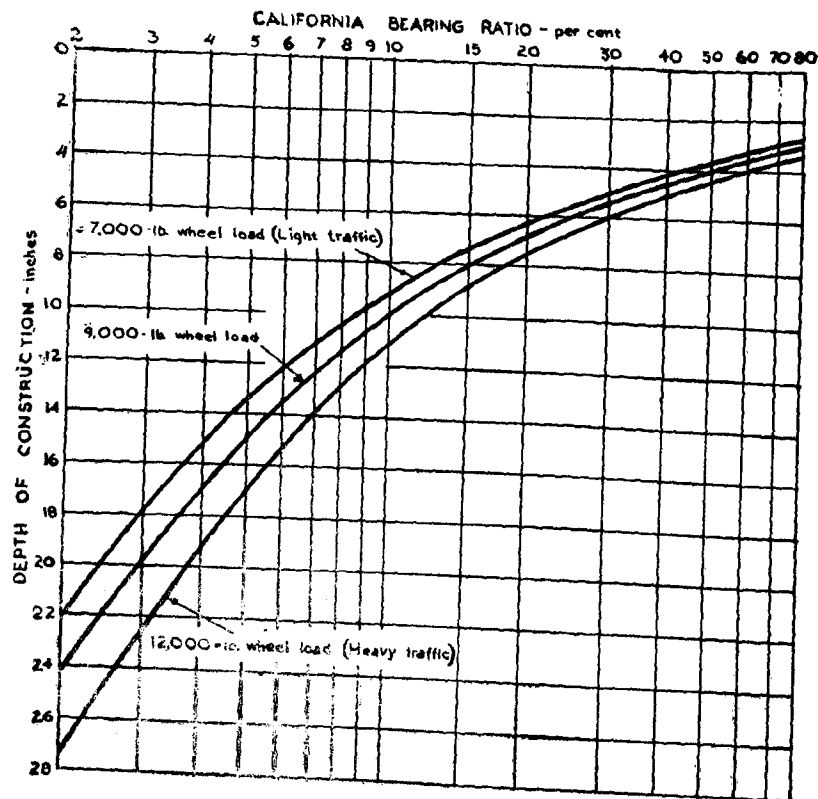
The plate bearing test is one of the tests for designing the thickness of the pavement. But this test requires bulky field equipment to supply dead load reaction and has a lengthy test procedure. The most important method, therefore, is to employ a small scale bearing test known as C.B.R. test as it is a simple and a speedy test. Tests are carried out on a compacted sample, to be tested after soaking for four days so as to produce a severe subgrade condition comparable to the one met with in the field during monsoon and rainy days.

3. DESIGN

3.1. The tentative design of the pavement is arrived at after taking the following safe values of C.B.R., which are arrived at after several laboratory tests for different layers for a wheel load of 9,000 lb :

- (a) Sandy soil subgrade having C. B. R. = 3.5 (Table 5)

- (b) Sandy soil locally available from Survey No. 274 having C. B. R. = 13 (Table 6)
- (c) Mixture of pure sand from the bed of the nullah and the sandy soil from Survey No. 274 in proportion 2 : 1 having C. B. R. = 20 (Table 4)



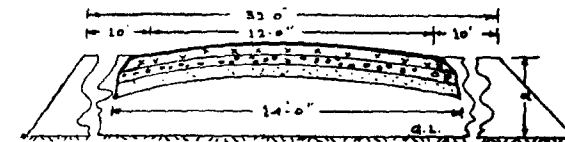
DESIGN CURVES FOR ROADS, CALIFORNIA BEARING RATIO METHOD.

PAGE NO. 407 OF SOIL MECHANICS FOR ROAD ENGINEERS VOL
PUBLISHED BY D. S. I. R. LONDON

Fig. No. 20.7,

The total thickness of the pavement for a subgrade having C. B. R. of 3.5 comes to 18 in. (as given in the attached Fig. No. 20.7* taken from page 407 of "Soil Mechanics for Road Engineers", published by D.S.I.R. London).¹

The following design is recommended on the basis of various C. B. R. values of the proposed layers between the wearing coat and the subgrade soil, Fig. 3.



CROSS SECTION FOR THE STABILISED ROAD
AS DESIGNED BY C.B.R. METHOD ON
DABHOI-BODELI ROAD MILE 15/7 TO 16/3.

NO. LAYERS.	DESCRIPTIONS.	THICK- NESS INCH.	C.B.R.	DRY DENSITY in lb/cu ft	OMC %
1	ASPHALT CARPET	3/4	-	-	-
2	METAL LAYER 1 1/2 TO 2" SIZE.	5/4	100	-	-
3	MIXTURE OF PURE SAND AND SANDY SOIL IN RATIO 2:1	3	20	117	10.0
4	SANDY SOIL FROM SURVEY NO.	9	15	102.6	12.5
5	FILLING LAYERS ROLLED AT PROCTOR DENSITY	d	3.5	94.5	17.0
C	NATURAL SUBGRADE	-	3.5	94.5	17.0

Fig. 3.

Requirements of the total thickness on subgrade of 3.5 per cent C. B. R. is equal to 18 inches.

* Reproduced with due acknowledgment.

Type of material	Thickness of layers	Remarks
1. First layer, sandy soil locally available from Survey No. 274 having C. B. R. = 13 per cent	9 in.	These layers should be properly compacted at densities and water content as mentioned in Table 6.
2. Second layer of mixture of pure sand from the bed of nullah near Vega village and sandy soil locally available from Survey No. 274 having C.B.R. = 20 per cent	3 in.	
3. Wearing coat of metal	5½ in.	
4. Asphalt carpet surfacing	¾ in.	

The above design was adopted for the proposed Dabhoi-Bodeli soil stabilized road. Depending upon the availability of the materials, the economical lead and site condition, slight modification will have to be made if there is appreciable change in the wheel load. Success of the experimental stabilized road depends on the proper mixing of the materials, proper compaction in suitable layers, and proper watering at optimum moisture as already mentioned.

4. CONSTRUCTION

4.1. Preliminary

The longitudinal section of the subgrade for the proposed length of the road in mile 15/7 to 16/3 was first taken up and the formation level was decided on the map. 100 ft wide land was acquired and boundary stones were fixed at every 220 ft on either side of the width.

4.2. Finishing the Surface of the Subgrade

Vegetables were removed with their roots by digging the subgrade about 6 in. deep. Then pegs were fixed at every 25 ft intervals in the centre line of the road. Markings were made on each peg showing the level of the formation of the road to make it easy for the labourers to do filling or cutting at the position of the pegs.



Photo 1.
Field density test



Photo 2.
California bearing ratio test



Photo 3.
Liquid limit and plastic limit test

A service path 10 ft wide was prepared with its centre line 29 ft on left hand side of the centre line of the main road.

4.3. Organization

As the work was to be done departmentally, a base camp was set up near Vega village. This camp had the field laboratory, stores and accommodation for the staff.

4.4. Field Control

Base Course	Maximum dry density in lb per cu. ft.	Optimum moisture content
		Per cent
A. Subgrade (where filling is done)	96	17.6
B. First layer 9 in. thick	102.6	12.6
C. Second layer 3 in. thick	117	10
D. Camber	1 in 48	

4.5. Machinery Used

The following machinery was used on this work:

- (i) Sheepsfoot roller - $4\frac{1}{2}$ tons
- (ii) Tractor
- (iii) Tracto-Tanker
- (iv) Roller (smooth-wheeled)
- (v) Truck

For details of the above items, see Appendix 5.

4.6. Preparing the Formation

As the ground was uneven, it was necessary to do the earth-work in filling and also in cutting according to the designed longitudinal section upto the formation level which was 18 in.

Photo 4.

Supervising the rolling work



Photo 5.

Dressing the sub-grade



Photo 6.

Measured quantity of earth



below the road level. The C.B.R. tests were carried out on the subgrade sample of black cotton soil at 90 per cent of the Proctor density of 105 lb per cu. ft., i.e., at 94.5 lb per cu. ft. So, where the formation level was in cutting, it was not necessary to roll it as the density of the subgrade was not distributed but rolling was necessary in case of filling to get the dry density of 94.5 lb per cu. ft.

As the earth was to be rolled at optimum moisture content, it was necessary that a measured quantity of earth should be dug out from the borrow pits and spread in 13 ft width of the road. For this the dimensions of the borrow pits were fixed as 8 ft by 12 ft 6 in. giving 100 sq. ft. These borrow pits from which earth was taken for the central 13 ft width of the road were made all along one side of the road and the borrow pits for the side berms were made along the other side of the road.

The following example explains how to arrive at the quantity of water to be added above the hygroscopic moisture content to the measured quantity of earth spread.

Example

- | | |
|-----------------------------|--------------------|
| 1. Optimum moisture content | 15 per cent |
| 2. Hygroscopic moisture | 10 per cent |
| 3. Dry density | 120 lb per cu. ft. |

To find

1. The bulk density.
2. Extra quantity of water to be added for optimum moisture content.

Solution

$$\text{Wet density} = \frac{(100+10) \times 120}{100}$$

$$= 132.00 \text{ lb per cu. ft.}$$

$$\therefore \text{Extra quantity of water required} = \frac{132(15-10)}{110} = 6.$$

$\therefore$ 6 lb of water are required to be added to one cubic foot of the undistributed soil.

TABLE 1
Showing the properties of the subgrade soil samples from mile No. 15/7 to 16/3

Lab. No.	Particulars			Per cent passing							Liquid limit	Plastic limit	Plasticity index	Maximum proctor density in lb per cu. ft.	Optimum moisture content percentage	Classification
	Trial pit	Mile No. Chainage in ft	Course depth in ft	Coarse sand (2-2 mm.) percentage	Fine sand (2-05 mm.) percentage	Silt percent- age (.005 to 0.05 mm.)	Clay percentage (00. > .005 mm.)									
993	1	84,000	1	10	35	32	22	39.00	17.66	21.34	105.4	16.3	Clay loam			
994	2	84,750	1	15	35	34	16	43.50	21.10	22.40	105.2	17.6	Clay loam			
995	3	85,300	1	11	36	30	23	41.40	19.70	21.70	104.3	16.8	Clay loam			
996	4	86,650	1	9	33	35	23	38.60	18.90	19.70	106.07	10.2	Clay loam			
997	5	86,450	1	12	32	35	21	42.90	19.70	23.20	107.1	17.3	Clay loam			

TABLE 2
Showing the properties of the sandy soil locally available from Survey No. 274

Lab. No.	Particulars			Per cent passing				Liquid limit	Plastic limit	Plasticity index	Maximum proctor density in lb per cu. ft.	Optimum moisture content in percent-age	Classification
	Trial it	Mile No. Chainage in ft	Depth in ft	Coarse sand percent-age (2-2 mm)	Fine sand percent-age (2-06 mm)	Silt percent-age (05 to .005mm)	Clay percent-age (.005 mm)						
1107	No. 1	84,900 Survey No. 274	1-2	42	28	23	7	—	—	Non-Plastic	108.4	16.0	Sand
1108			3-5	54	31	11	4	—	—	"	114.2	12.6	Sand
1109			6-7	52	33	10	5	—	—	"	113.7	13.1	Sand
1110	No. 2	84,875 Survey No. 274	1-2	44	25	25	6	—	—	"	107.3	17.3	Sand
1111			3-5	56	30	9	5	—	—	"	115.2	13.0	Sand
1112			6-7	55	32		4	—	—	"	114.7	12.4	Sand

TABLE 3

Sieve analysis test on pure sand

Sieve Nos. (BSS)	Per cent passing		
	Sample No. 1	Sample No. 2	Sample No. 3
$\frac{1}{2}$	100	100	100
$\frac{3}{8}$	100	100	100
$\frac{1}{4}$	98	99	100
$\frac{3}{16}$	97	98	98
7	91	92	95
14	77	77	80
25	47	44	41
52	20	18	10
100	10	10	3
200	3	3	1

TABLE 4

B.S. sieve size	Per cent Passing										Remarks
	Specified size distri- bution for 3/8 in. maximum base		Natural sand from the bed of the nullah A	Sandy soil from Survey No. 274 B	Sand		Sandy Soil		Sand	Sandy Soil	
					A	B	A	B			
	Limits	Average	4 : 1	6 : 1	2 : 1						
1	2	3	4	5	6	7	8	9			
3/8	100	100	100	100	100	100	100		The mixture of A and B in the proportion 2 : 1 was used in 3 in. layers on the work as the same gives C.B.R. value of 20 and it also satisfies percentage passing through 200 B.S. sieve.		
3/16	80-100	90	98	100	98	98	99				
7	50-80	65	92	100	94	93	95				
14	40-65	53	77	96	81	80	83				
52	20-40	30	18	63	27	24	33				
200	10-25	18	3	27	8	6	11				

TABLE 5

Existing subgrade

The existing subgrade of Dabhoi-Bodeli Road mile 15/7 to 15/3 was tested for its natural density and C.B.R. Test at the existing density was carried out on remoulded sample to know its bearing capacity in the worst condition. The results are tabulated as under :

Lab. No.	Particulars chainage	Field density in lb per cu. ft.	Moisture (hygroscopic) per cent	0.9 of Proctor density at which C.B.R. is done	O.M.C. percentage	C.B.R. percentage soaked (4 days)	Swelling percentage	Remarks
1	2	3	4	5	6	7	8	9
993	84,000	99.37	4.70	94.86	16.3	3.56	0.29	C.B.R. was carried out on representative samples at 0.9 of max. Proctor density & at O.M.C.
994	84,750	98.79	5.43	94.68	17.6	3.45	0.27	
995	85,300	97.83	6.30	93.87	16.8	3.51	0.27	
996	85,650	99.34	5.82	95.49	16.2	3.33	0.26	
997	86,450	98.87	15.92	96.39	17.3	3.66	0.32	

From the above Table it is seen that the subgrade is clayey and poorly compacted. Though the field density is higher than the 0.9 of Proctor density, the C.B.R. test is carried out at 0.9 of the Proctor density, i.e., for the worst condition.

TABLE 6
Locally available sandy soil in Survey Field No. 274

The material from two pits of survey number 274 was tested for its natural density and C. B. R. test at the existing density was carried out on moulded sample in worst condition to know its bearing capacity. The results are tabulated as under:

Lab. No.	Particulars			Field density in lb per cu. ft.	Moisture percent-age	0.9 of proctor density lb per cu. ft.	O. M. C. Percent-age	C. B. R. values soaked 4 days	Swelling percent-age	Remarks
	Trial pit	Mile No.	Location							
1	2	3	4	5	6	7	8	9	10	11
1	No. 1	84,900 ft	Survey No. 274	113.7	8.11	97.56	16	13		
2				110.8	8.70	102.78	12.6			
3				107.7	7.58	102.33	13.1			
4	No. 2	84,875 ft	Survey No. 274	112.6	8.51	95.57	17.3	13.3		C. B. R. was carried out on representative samples at 0.9 of max. Proctor density at O. M. C. in both the cases.
5				108.7	7.39	103.68	13			
6				111.3	9.51	103.23	12.4			

The field density is much higher than 0.9 of proctor density. Even then the C. B. R. test is carried out at 0.9 of the proctor density to have the C. B. R. value in the worst condition.

Rolling was carried out with the sheepfoot roller to give the dry density of 94.5 lb per cu. ft. To find out the minimum number of passes of the roller to give the maximum density of 94.5 lb per cu. ft., an experiment was conducted to give various different dry densities at different number of passes of the roller, vide Table 7 given below and Fig. 4. A graph was plotted of the dry densities against number of passes and it was decided from the graph that with eight passes the compacted earth would give a dry density of 97.2 lb per cu. ft.

TABLE 7

Results of the experiment on subgrade showing the number of passes and dry density obtained

Sl. No.	No. of passes	Dry density in lb per cu. ft.
1.	8	97.2
2.	16	104.00
3.	24	107.00
4.	32	108.00

The optimum moisture content for the subgrade of black cotton soil was 17.6 per cent. Actually this quantity of water is so high that it was not possible to roll the earth layer immediately and it was necessary to wait for minimum twelve hours after spreading the water. So another test was carried out on a 100 ft length in such a way that the percentage of water to be added was varied from 10 to 17 per cent in five different layers of 9 in. each (loose) keeping the number of passes constant. The results of the test are shown in Table 8 and Fig. 5.

TABLE 8

Sl. No.	Percentage moisture added	Dry density in lb per cu. ft.	No. of passes
1.	10	109.5	8
2.	12	110	8
3.	14	112	8
4.	16	113.6	8
5.	17	113.6	8

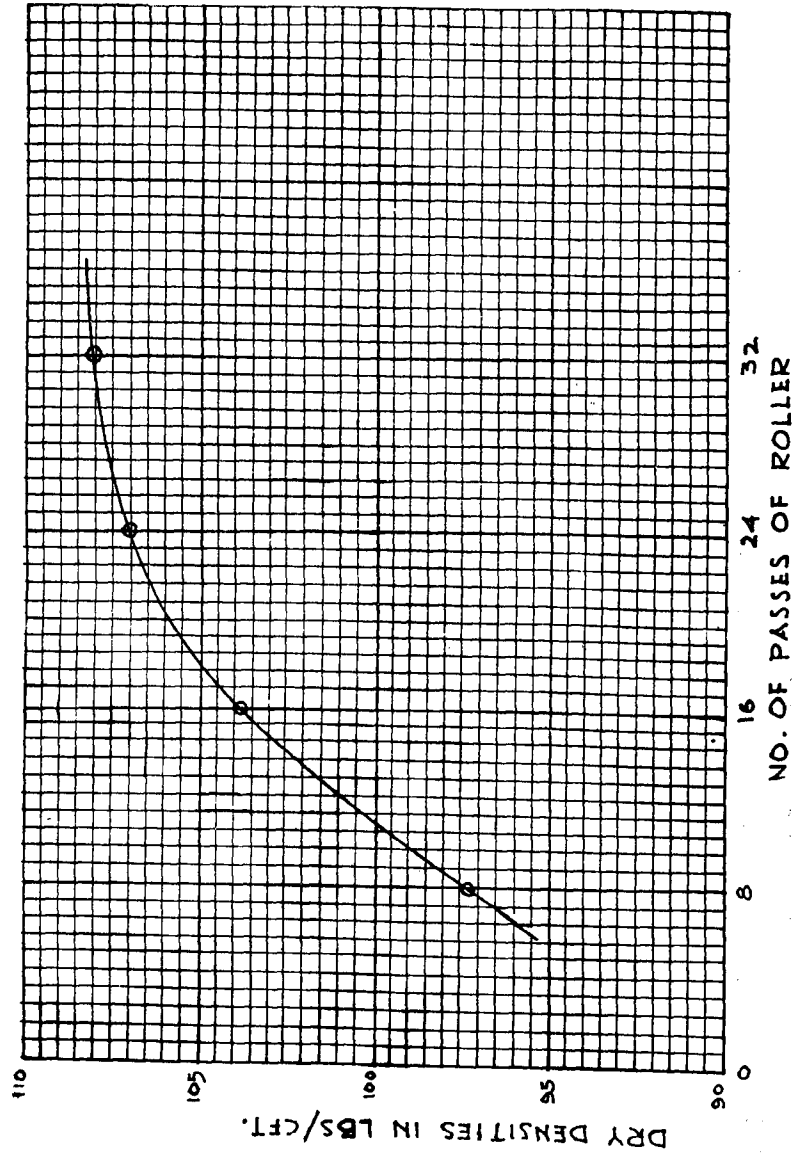


Fig. 4. RESULTS OF TEST ON SUBGRADE ROLLED AT O.M.C.

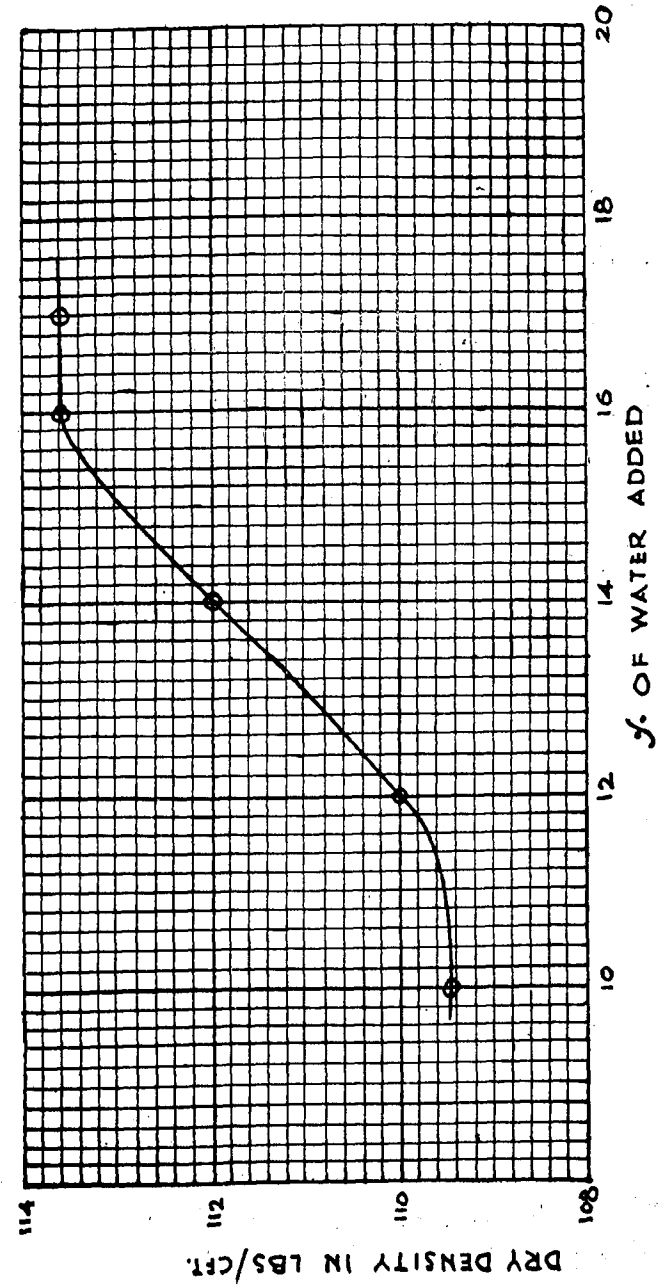


Fig. 5. RESULTS OF TEST ON SUBGRADE WITH EIGHT NUMBER OF PASSES.

It is seen from Fig. 3 that even 10 per cent moisture with eight number of passes gives a very high density. So, only 10 per cent of water was added and the earthwork was completed in 9 in. layers laid loose compacted to 6 inches.

The results of compaction obtained at optimum moisture content at different places on the subgrade are given in Appendix 1.

4.7. First Layer on Subgrade

This layer was laid in two stages. In the first stage, the sandy soil was spread in 7 in. thickness loose. Calculated quantity of water was spread over the layer. A test was also carried out on the first 100 ft length of road to ascertain the minimum number of passes to get the maximum dry density of 102.6 lb per cu. ft. The results of the tests are shown in Table 9 and Fig 6.

TABLE 9

Sl. No.	No. of passes of the smooth roller (8 ton)	Dry density in lb per cu. ft.	Designed dry density and optimum moisture content
1.	4	100.3	
2.	8	111.7	
3.	12	122.7	102.6 lb per cu. ft. at
4.	16	127.0	12.6 per cent
5.	20	131.5	
6.	24	135.1	
7.	28	138.7	
8.	32	139.0	

The first layer of 7 in. was thus compacted to $5\frac{1}{2}$ in. with 8 number of passes.

In the second stage, the sandy soil was spread 5 in. thick in loose state and it was rolled to $3\frac{1}{2}$ in. with 8 number of passes, after the addition of the required quantity of water.

After completing the first layer of 9 in., the tractor was run on the whole length of the road to develop uneven surface with the help of the plate projections, to give proper bond to the second layer 3 in. thick.

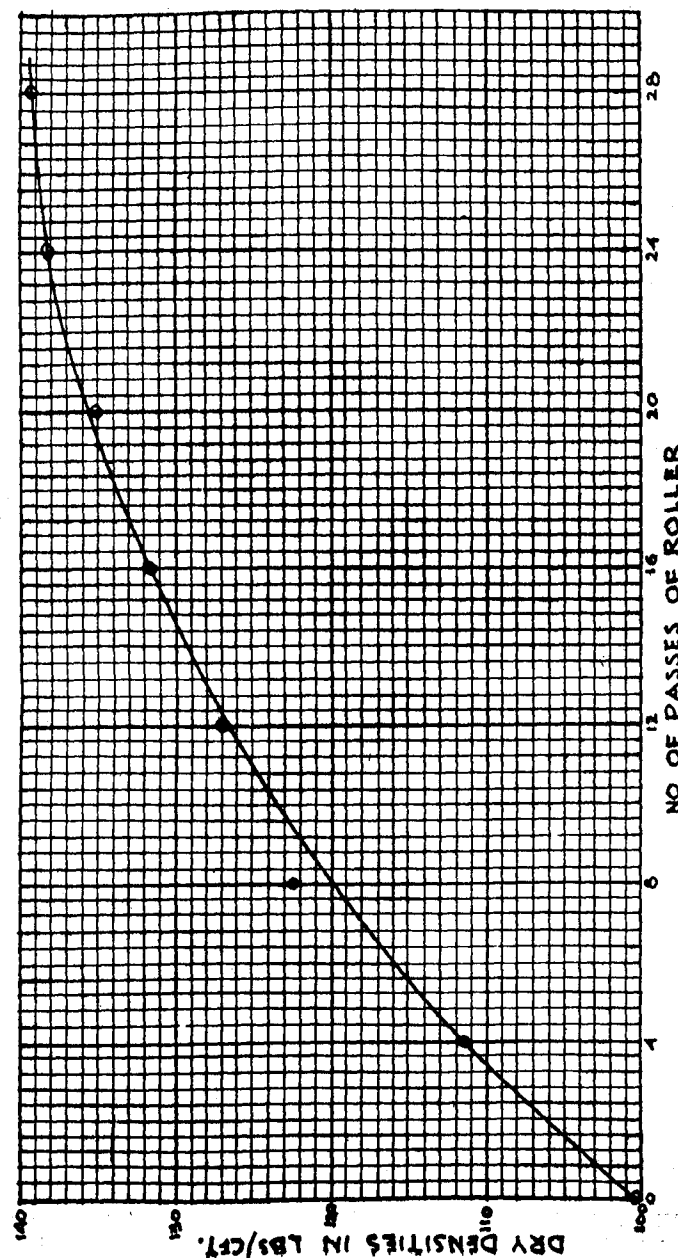


Fig. 6. RESULT OF TEST ON SANDY SOIL ROLLED AT O.M.C.

The results of compaction obtained on the first sandy layer of 9 in. thickness after rolling it at optimum moisture content are given in Appendix 2.

4.8. The Second Layer

The second layer was 3 in. thick and it was designed as a mixture of pure sand from the bed of the nullah running nearby and the sandy soil available locally from the Survey No. 274 in the proportion of 2:1. The mixing of these two materials was carried out by "windrow" method. In this method the quantity of each material was calculated for every 100 ft length and then two heaps were made at either end of the 100 ft length, each heap consisting of half the quantity of pure sand with half the quantity of sandy soil over it. The labourers used to take the mixture from one side of the heap with spades. Spreading of this mixture was started from either end of the 100 ft length to economise in leads. Thus each heap was spread in the length of 50 ft in approximately 6 in. thick loose layer which was ultimately compacted to 3 inches. Both the first and the second layers were rolled to a camber of 1 in 48. An experiment was conducted on this layer also on a 100 ft long strip to determine the most economical number of passes to give the maximum dry density of 117 lb per cu. ft. at 10 per cent optimum moisture content. The results of the test are shown in Table 10 and Fig 7.

TABLE 10

Sl. No.	No. of passes of 8-ton smooth roller	Dry density in lb per cu. ft.	Remarks
1.	4	107.50	The roller was too heavy for 3 in. thick strip as indicated by cracks and removal of some cakes.
2.	8	109.50	
3.	12	115.20	
4.	16	116.00	
5.	20	118.20	
6.	20	118.20	
7.	24	119.10	

It was decided from Fig. 7 to roll this layer with 20 passes at optimum moisture content.

The results of compaction obtained on the second 3 in. layer of mixture of sandy soil and pure sand at optimum moisture content are given in Appendix 3.

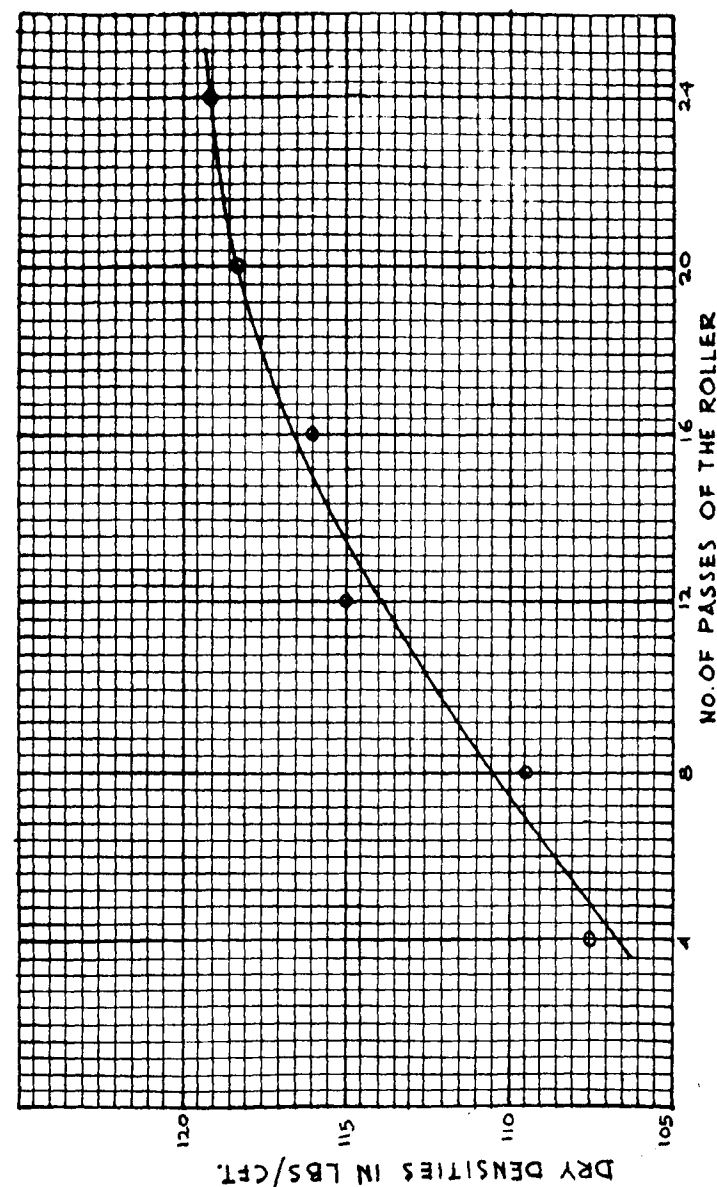


Fig. 7. RESULTS OF TEST ON MIXTURE ROLLED AT O.M.C.

5. COST

The general details of cost are given in Appendix 4. Cost of 12 in. stabilized base course only is as follows :

Sand	Rs 355 8 0
Sandy soil: Departmental Rs 307/8/0 Piece worker Rs 985/-/-	1,292 8 0
Labour for watering, etc., proportionately for 28 days and on this course only	242 0 0
Fuels for roller, truck, tanker, etc., proportionately for 28 days and on this layer only	321 0 0
Establishment - 24 days excluding that for earth layers	532 0 0
Spreading sandy soil, etc.	484 8 0
	<u>Rs 3,227 8 0</u>

$$\begin{aligned}\text{Rate of work done} &= 4 \times 660 \text{ ft} \times 12 \text{ ft} \times 1 \text{ ft} \\ &= 34,320 \text{ cu. ft.}\end{aligned}$$

$$\text{Rate per 100 cu. ft.} = \frac{3227.50 \times 100}{34320}$$

$$= \text{Rs } 9.69 \text{ per 100 cu. ft.}$$

$$\text{or Rs } 9.69 \text{ per sq. ft. (as the base course is 12 in. thick).}$$

$$\begin{aligned}\text{Cost of the extra } \frac{3}{4} \text{ in. thick metal used above the conventional thickness of } 4\frac{1}{2} \text{ in. of the metal} &= \frac{3}{4} \times \frac{1}{12} \times 100 \times \frac{75}{100} \\ &= \frac{225}{48} = \text{Rs } 4.69\end{aligned}$$

$$\therefore \text{Rate of the base course} = 9.69 + 4.69$$

$$= \text{Rs } 14.38 \text{ per 100 sq. ft.}$$

The cost of the brick soling ($4\frac{1}{2}$ in. thick brick on edge) used in the conventional type design in P. W. D., vide Fig. 1

$$= \text{Rs } 25 \text{ per 100 sq. ft.}$$

$$\begin{aligned}\therefore \text{Nett saving in the base course below the metal surface of } 4\frac{1}{2} \text{ in. thickness} &= \text{Rs } 25 - \text{Rs } 14.38 \\ &= \text{Rs } 10.62 \text{ per 100 sq. ft.}\end{aligned}$$

$$\text{i. e. } \frac{10.62}{25} \times 100 = 42.48 \text{ per cent saving per 100 sq. ft.}$$

Note: The work has been recently completed and the behaviour of the road under traffic to ascertain life thereof is being watched.

ACKNOWLEDGMENT

The Author is grateful to Shri B. K. Choksi, Superintending Engineer, South Gujarat Circle (R & B), Baroda, and Shri I. K. Naik and Shri M. M. Patel, Executive Engineers, Baroda Division (R & B), Baroda, for their valuable help, suggestions and facilities given for construction of this work at various stages.

The Author is also indebted to Shri O. H. Patel, Professor of Civil Engineering, M. S. University, Baroda, for his valuable help and guidance during the design work and testing facilities in the laboratory and to Shri J. M. Parekh, Deputy Engineer, Dabhoi Subdivision, Dabhoi for his help and facilities given during execution of this work.

REFERENCES

1. "Soil Mechanics for Road Engineers" - Department of Scientific and Industrial Research, Road Research Laboratory, London.
2. "Elements of Soil Mechanics" - By Capper and Cassie.

Appendix 1

Results of Compaction

Dry densities obtained on the subgrade after rolling it at O.M.C. at the places of embankment.

Mile No.	Chainages in feet	Dry density	Mile No.	Chainages in feet	Dry densities in lb per cu. ft.
1	2	3	4	5	6
15-7	83820			85375	
		105.2			101.25
	83900			85475	
		109.6			103.70
	84000			85575	
		108.7			99.9
	84100			85675	
		106.0			97.6
	84200			85775	
		108.0			101.3
	84300		16-2	85875	
		113.6			104.7
	84400			85975	
		108.8			100.8
16-0	84500			86075	
		110.3			102.5
	84600			86175	
		101			102.9
	84660			86275	
		107.8			103.6
	84750			86375	
		108.5			99.4
	84815		16-3	86460	
		105.7			
16-1	84925	Culvert portion			
	85150				
		107.9			
	85290				
		107.9			
	85375				

Appendix 2

Results of Compaction

Dry densities obtained on the first sandy layer of 9 in. thickness after rolling it at optimum moisture content.

Mile No.	Chainages in feet	Dry densities in lb per cu. ft.	Mile No.	Chainages in feet	Dry densities in lb per cu. ft.
1	2	3	4	5	6
15-7	83320			85450	
		116.1			112.6
	83530			85550	
		110.6			114.3
	83950			85650	
		112.5			114.0
	84050			85750	
		111.2			114.5
	84150		16-2	85850	
		112.2			113.3
	84250			85950	
		109.1			115.3
	84350			86050	
		105.3			113.0
	84450			86150	
		102.9			114.2
16-0	84550			86250	
		104.6			114.9
	84650			86350	
		115.5			115.2
	84750		16-3	86450	
		116.3			
	84850				
		110.0			
	84950				
16-1	85150	Culvert portion			
		112.2			
	85250				
		113.1			
	85350				
		110.9			
	85450				

Appendix 3

Results of Compaction

Dry densities obtained on the mixture of sandy soil and pure sand in the layer of 3 in. after rolling it at optimum moisture content.

Mile No.	Chainages in feet	Dry densities in lb per cu. ft.	Mile No.	Chainages in feet	Dry densities in lb per cu. ft.
1	2	3	4	5	6
15-7	83820			85350	
		119.2			113.8
	83950			85450	
		114.00			114.2
	84050			85550	
		113.1			115.7
	84250			85650	
		114.9			114.3
	84350			85750	
		115.1			117.2
	84450		16-2	85850	
		116.3			116.1
16-0	84550			85950	
		114.7			114.2
	84650			86050	
		115.8			113.9
	84750			86150	
		117.3			114.5
	84850			86250	
		115.7			115.3
	84950			86350	
		116.3			116.9
	85050		16-3	86450	
		114.5			
16-1	85150				
		113.9			
	85250				
		114.2			
	85350				

Appendix 4

COST

I. Materials Purchased by Requisitions

	Date	Rs	As	Ps
Pegs	15- 1-57	8	0	0
Oils	11- 2-57	83	8	0
Lanterns	11- 2-57	10	14	0
Oils	11- 2-57	27	5	0
Spirit	14- 2-57	20	15	0
Petrol	10- 3-57	24	8	0
Oils	10- 3-57	108	10	0
Oils	10- 3-57	78	0	0
Oils	10- 3-57	35	14	0
Oils	29- 3-57	33	0	0
		430	10	0
		Rs Np		
		i. e. 430.62		

Payments made.

Materials purchased in anticipation on			Rs	As	Ps
Date	Requisition	No.			
24- 4-57		159/7-3-57	84	5	0
24- 4-57	"	No. 169	86	10	0
3- 4-57	"	No. 1/9-4-57	119	14	0
3- 4-57	"	No. 2/9-4-57	303	0	0
			1024	7	0
			Rs Np		
			i. e. 1024.44		

II. Establishment

	Rs	Np
November 1956	106.00	
December 1956	206.00	
January 1957	330.06	
February 1957	661.81	
March 1957	745.07	
April 1957	707.63	
	2756.57	
	Rs Np	
	i. e. 2756.57	

III. Rate List

	Rs	As	Ps	
1. Sharpening of tools	24	12	0	
2. Repairs to roller	20	8	0	
3. Constructing temp. shed	321	15	0	
	367	3	0	Rs Np
				i. e. 367.19

IV. M. Rolls

Date	to	Date	Rs	As	Ps	
10-11-56	„	20-11-56	99	8	0	
21-11-56	„	20-12-56	127	8	0	+ 80 0 0
21-12-56	„	20- 1-57	64	8	0	
21- 1-57	„	20- 2-57	2124	0	0	
21- 2-57	„	20- 3-57	1386	0	0	
21- 3-57	„	20- 4-57	3003	12	0	
21- 4-57	„	20- 5-57	617	4	0	
			7506	8	0	Rs Np
						i. e. 7506.50

V. Work done by Rate List

		Rs	As	Ps	
1. R. A. Bill	Earthwork	634	5	0	
	Sandy soil	709	5	0	
Final	Earthwork	387	0	0	
	Sandy soil	285	0	0	
		2015	10	0	Rs Np
					i. e. 2015.62

	Rs	Np
I. Purchase by requisition	1024.44	
II. Establishment	2756.57	
III. Rate list for shed etc.	367.19	
IV. M. Rolls	7506.50	
V. Work done by piece worker	2015.62	
	13669.72	

Appendix 5

DETAILS OF MACHINERY USED

The following machinery was used on this work, the details of which are given below :

- (i) **Sheepsfoot Roller**—4½ ton weight.
Nob pressure—504 lb per sq. in.
Use—Used to roll the subgrade soil in filling.
- (ii) **Tractor**—Diesel Cletrac Power (U. S. A.), Cleve & Tractor Company.
Size—4¾ × 5¼.
Engine No.—S. 386347.
Model—DR X B.
Fuel—Diesel oil.
Lubricating oil—Mobil oil, C. oil, Gear oil, etc.
Horse power—60.
- (iii) **Tracto - Tanker**—Allis Chalmer.
Tractor No.—BYS 1974.
Fuel—Powerine.
Lubricating oil—Mobil oil, C. oil, Gear oil, etc.
Horse power—20.
Use—Used with light bullock roller.
- (iv) **Roller** (smooth wheeled)—Marshall Sons & Co. Ltd., Gainsborough, England.
Roller type—U. D.
Size—8 tons.
Roller No.—90900.
Fuel—Diesel.
Lubricating oil—Mobil oil, C. oil, Gear oil, etc.
Use—Compaction of the first 9 in. layer and the second 3 in. thick layer.

(v) **Truck**

No.—4462.

Fuel—Petrol.

Lubricating oil—Mobil oil, C. oil, Gear oil, etc.

Miscellaneous — Grease, Brake oil, Battery charging
solution, Distilled water.

Use—To bring the pure sand from nullah bed and stack
at the site.

INFORMATION SECTION

"ADHESION BETWEEN BITUMINOUS BINDERS AND ROAD STONES"

By

C. G. SWAMINATHAN*

AND

K. P. NAIR*

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1. INTRODUCTION

It is well-known that the major factor involved in the successful performance of a bituminous road is that of adhesion. Although nearly 90 per cent of the material that goes to make up the structure of the road surface is built by the aggregates, it is obvious that, but for the binding effect of the bituminous material, the surface will not be able to withstand the rigours of traffic. Unless the adhesive bond is good, the aggregates will get dislodged and the road will start failing.

In the process of adhesion there are two stages, namely, firstly the coating or wetting of the aggregate and secondly the development of the adhesive strength. If either is absent, the aggregates will only be loosely knit instead of forming a tight mat. For example, water may coat the aggregates but two stones will not adhere to each other by a thin film of water alone. For this purpose, viscosity for cohesion and other characteristics of the

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liquid are required in the binding medium. These are possessed only by bituminous materials such as road tars and asphaltic bitumens.

Generally, if the binder is stripped off the aggregate, with a resultant failure of the road, assuming that the proper grade and type of binder and aggregates were employed, the reasons could be attributed to one or more of the following causes:

- (a) Effect of water,
- (b) Overheating of the binder or the aggregate or both, and
- (c) Presence of dust on the aggregates.

In the first case, it is universally known that the affinity between stone and water is better than that between the binder and stone. So in the prolonged presence of water there is a tendency for the binder to be stripped from the aggregate and this tendency is more when the binder is fresh and has not developed the full viscosity to secure a strong bond.

In the second case, overheating of the aggregate or the binder above the specified temperature results in such an enormous increase in the viscosity of the binder at the interface, that the aggregates are not properly and uniformly wetted. Further, heating makes the binder brittle, as a result of which the film of binder ruptures, thus enabling entry of water to complete the separation of the binder from the aggregate.

In the third case, the binder instead of coating the aggregates coats only the fine dust covering the stone resulting in a subsequent peeling off of the coated dust from the stone by the action of traffic and water.

Even if the aggregates were free from dust and the materials were heated and laid as per specifications, the ingress of water subsequently could bring damage to the surface by stripping the binder off the aggregate, leaving the stones bare with a resultant instability of the road metal. It is obvious that this type of failure in a bituminous surface is more pronounced and widespread in open-textured surfaces such as single-sized premix carpets or surface dressings than in dense graded mixtures. For, in the latter case the mixture is so dense that the permeability to water is low whereas in the former the individual voids are sufficient to retain water for quite a long duration, thereby causing the slow removal of the binder from the aggregates. This water which finds its way into the bituminous road struc-

ture could be either the underground water which rises up due to capillary action when the water table goes up or it could be due to rain water which penetrates into the interstices and is retained for some time. Further, if the construction is done during rainy season the aggregates will be wet with the result that the binder will not coat the stones properly. Also, if the rain falls immediately after construction, when the surface is quite fresh, owing to the low viscosity of the binder at that stage it is dislodged.

Even if the construction is done during dry weather, there are possibilities of stripping taking place subsequently owing to rainfall. Some aggregates resist the action of water better than others. In some cases, by adding some surface active agents, this resistance is increased. Some binders coat the stones better than others. The reasons for these have been investigated by a number of people using different methods of testing.

And from those investigations, certain salient points regarding the nature, the how and why, and the various factors which influence stripping, have come to light and they form the subject matter of this note.

2. PRINCIPLE OF ADHESION

The mechanism of adhesion between a stone aggregate and a bituminous binder is still not very clear, though quite a few theories have been put forth. According to Sacher¹ adhesion is due to residual valencies conditioned by the presence of certain active atom groups in the molecules. Also, it is stated that when a solid is wetted by a liquid, adsorption occurs at the surface followed by chemical reaction between the adsorbed material and the constituents of the solid phase. According to Shaw³ minerals adsorb air to a remarkable degree and this is a cause for the non-wetting with oil, hydrophobic aggregates having a loosely held oxygen film as against hydrophilic ones which have a tightly held film. Thelen⁴ has given a likely reason for stripping in the presence of water when there is no film break. According to him the dissolved air bubbles in water attempt to attach themselves to the asphalt surface and when they do so, the asphalt spreads over the interface between the air bubbles and the water, thus enclosing the air. On the drying of the surface, the energy potentials are reversed and the asphalt film enclosing the bubbles tends to pull back leaving a hair crack.

In all these theories the effect of surface texture and the energies at the contact surface are not given due importance

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From a physical standpoint of view a liquid can be said to wet a solid when the forces of attraction between the molecules of the liquid and the solid are greater than the forces of attraction of the molecules of the liquid for each other. After wetting, the liquid becomes an adhesive if it is converted into a tenacious mass by cooling, evaporation or other processes. When the binder is hot its viscosity is low and it wets the aggregate and afterwards, on cooling, the viscosity increases with a resultant increase in its resistance to the shearing action. Thus the effect of adhesion is a combined effect of surface tension, adsorption and other properties at the solid-liquid interface.

When a drop of liquid is placed over a solid plane some liquids spread out and some retain a spherical shape, the reason being due to surface tension. The spreading force of the liquid along the solid plane is governed by the surface tension at the solid air phase, namely, γ_{sa} , the interfacial tension between the solid and the liquid, namely, γ_{sl} and the surface tension of the liquid air phase, namely, γ_{al} . If α is the angle of contact, at equilibrium conditions:

$$\gamma_{al} \cos \alpha = \gamma_{as} - \gamma_{sl}.$$

The equation will be $\gamma_{wl} \cos \alpha$

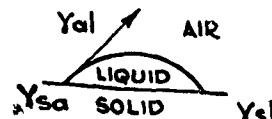
if instead of air the whole thing is immer-

sed in water. A lowering of the interfacial tension of the solid water interface (γ_{ws}) and a lowering of the solid liquid interface (γ_{sl}) would promote binder adhesion. The larger the difference the greater will be the tendency of the water to strip off the binder. $\gamma_{wl} \cos \alpha$ is known as the adhesion tension and in order that complete wetting may take place, the contact angle should become zero which happens when the surface tension and the interfacial tension of the liquid are lowered.

It has been found⁵ that the interfacial tension not only depends on the nature of the liquid but also on the surface texture of the stone. "The displacement of the binder involves a certain change in surface energy equal to the difference between the interfacial energies at the stone binder interface and the stone water interface".

Thus a reliable principle for the assessment of adhesion is quite possible based on the concept of the interfacial tension, although there are differences of opinion regarding the influence of surface tension in the phenomenon of adhesion.

But the exact determination of the surface tensions, etc., are not possible in the laboratory since the measurements with bitumens can only be made at high temperatures thereby entailing dif-



ficulties of measurement. Further, the contact angle is influenced by the area of contact, etc., with the result that the measurements can be made only on uniform well polished surfacing rather than on rough road stones. However, this method of approach does give an insight into the problem of adhesion to a certain extent. But for all practical purposes of evaluation of adhesion other methods are usually adopted.

3. METHODS FOR THE DETERMINATION OF ADHESION

Except where the fundamental properties such as surface tension, interfacial tension etc. are to be obtained, in all the other methods for the determination of the relative adhesive properties between a binder and an aggregate the first procedure that is adopted is to coat the aggregate with a known quantity of the bituminous binder and then to evaluate the amount of stripping of the binder that has taken place in the presence of water. Though the evaluation as such is based on visual inspection and results expressed in terms of percentages of stripping, the process of inducing the stripping varies with different methods in addition to the size and the qualities of the materials taken and the temperature conditions.

In the case of the Missouri boil test⁶ the coated aggregate is immersed in water and the water boiled. In another method the strength of an alkali solution (such as Na_2CO_3) as in the Riedel and Weber test⁷ or the pH values of the water solution causing stripping are noted down. In the case of the water immersion test⁸ the temperature of the water is the main factor. Since prolonged contact with water also causes the removal of the binder, the duration of immersion of the coated aggregate under water at specified conditions can also be a measure⁹. In another direct method the stones are kept over a film of binder on a plate, the whole thing immersed in water and afterwards the stones are pulled out and stripping evaluated¹⁰. Mechanical abrasion with the help of a rotating disc has also been resorted to for the evaluation of the amount of binder removed. The same can be found by means of a shaking machine also¹¹. Comparison of the strength values of two identical pre-mixed materials, one being dry and the other after complete soaking, have also been in vogue to express the effect of stripping¹². Of late, photo-electric methods¹³ and the use of radio active isotopes¹⁴ have also been attempted for the estimation of binder aggregate adhesion. Since stripping of the binder film on the road is caused not only by water but also by the mechanical effects of traffic, the number of passes of a given loaded wheel over the coated aggregates immersed in water under

specific conditions to cause a certain extent of disintegration can also be taken as a measure for stripping effects¹⁴.

But none of the above mentioned tests have been universally accepted. For, in majority of the tests the stripping is observed by visual inspection and as such the test data is influenced by personal errors and as such they remain as qualitative measures. In some of the so called quantitative methods, the test data is erratic and the reproducibility of the test data is not always good. In the other quantitative methods, stripping as induced in the laboratory is not always comparable to the actual field condition. Further, so many divergent set of variables such as size and surface texture of the stone, viscosity and nature of the binder, temperature condition at the time of coating, curing temperature and duration of immersion, etc., have to be taken into account in addition to the mode of inducing stripping and the exact evaluation of stripping. Sometimes, it so happens that two different tests, when conducted on the same materials, yield conflicting results. For, whereas one type of test gives the importance of only the effect of time, or that of temperature, another stresses only the chemical reaction at the interface, to the exclusion of the other important factors such as surface texture, viscosity, etc. So it can be seen that there is no universally accepted simple or definite method which would take into account all the aspects and enable an exact measurement of the adhesion between binder and stone—a value which would be correlatable to the actual service behaviour on the road.

4. FACTORS INVOLVED IN THE ADHESION OF BINDER TO STONE

From the point of view of the materials, the various factors which influence the adhesion of a stone to a binder can be studied under the different categories given below :

4.1. Aggregate

4.1.1. Chemical and mineralogical composition

It is customary to divide aggregates as possessing hydrophobic or hydrophilic characteristics based on their relative affinity to water. This property is sometimes related to the chemical and mineralogical composition. But according to Hallberg¹⁶, the classification of aggregates as hydrophobic and hydrophilic is untenable, since in his opinion adhesion will depend upon the combination of a certain binder with a certain aggregate. Saether¹ is of the opinion that hydrophobic pro-

perties of a mineral do not necessarily depend on basic character or chemical reactivity but in a complex manner upon the crystal structure and the nature of the ions of which it is composed. But, Ray and Thor¹⁷ are of the view that the major causes of hydrophobic and hydrophilic natures of the aggregates are the presence of water-soluble compounds and the degree of alkalinity inherent in the aggregate and the mineral composition. According to Knight¹⁸ and Oberbach¹⁹ adhesivity is more influenced by surface texture and other properties such as mechanical and crystalline structure than on chemical composition, though according to Knight¹⁸ the mineral composition affects the surface texture. Pfeiffer²⁰ has found that the presence of small quantities of iron at the surface of stones of high SiO_2 content causes less displacement of bitumen when in contact with water. And in a broad sense it can be said that it is easier to bind basic stones than acidic ones. This classification is based on the silica content, the stone being acidic for high silica contents. Based on this the stones giving poor adhesion (called hydrophilic) are syenites, granites, quartzites, porphyry and the stones which give fairly good adhesion (hydrophobic) are limestones, some basalts, hyperite, etc. A classification of the igneous rocks based on this is given below :

	Acid (high silica)	Intermediate	Basic (low silica)
Plutonic	Granite	Diorite	Gabbro
Dykes & sills	Quartz porphyry	Porphyrite	Dolerite
Volcanic	Rhyolite	Andesite	Basalt

However, it must be remembered here that many rocks passing under the same geological or petrological name may have very different mineralogical constitution and so it is not possible to give a simple geological classification of road stones based on adhesion.

4.1.2. Surface texture & crystalline structure

If, as said earlier, the chemical and mineral composition of the stone are important only in so far as their effect on surface texture is concerned, except for possible chemical reactivity at the interface, it would appear that the nature of the aggregate surface as influenced by the physical, mechanical and geometric properties of the stone is of greater importance. Texture can be classified as glassy, smooth, rough and crystalline. Though it is easier to coat and wet a smooth surface than a rough one, it is

obvious that once the latter is coated, the adhesive bond is better than that with the former, since a certain amount of mechanical interlocking is present thereby increasing the strength, and it has been found that with smooth, glassy and coarsely crystalline surfaces stripping of the binder is more pronounced than with the rough texture ones. The degree of crystallinity of a rock affects this texture and as between two rocks possessing similar chemical and mineralogical composition the one which gives a smooth fractured surface such as a cryptocrystalline flint is not so good as a crystalline quartzite from the point of view of bond. According to Knight¹⁸ the surface texture or rugosity and the grain size of a stone metal surface as disclosed under the microscope are of greater significance in determining adhesion than its mineral character. Lee⁶ has shown that differences in the adhesive properties recorded on the coarse minerals tended to disappear when the same tests were applied to highly polished surfaces of the same mineral.

It has been shown⁶ that the shape of the aggregate also has an influence on the relative displacement of the binder from the aggregate. Displacement begins sooner from stones with sharp edges than from rounded ones. As the binder films at the corners are very thin they easily get ruptured especially if the sharp edges get broken off. Similarly, dislodgment of the binder is more over proud spots and this may be expected in the case of too-coarsely textured surfaces. Irregular and rounded aggregates score an advantage over angular or flaky ones, but since good adhesive bond with rounded stones may pose a problem subsequently, irregular cubic chippings are preferable.

It is also said that a certain amount of porosity in the stone is desirable since it allows the bituminous material to penetrate into the pores and by giving an additional mechanical interlock improves the adhesion.

4.2. Binder

4.2.1. Nature

Between the two normally used binders, namely, tar and bitumen, tar has better wetting properties mainly because of the hydroxyl components in it. If all the phenolic components are removed from the tars by washing with alkali, the wetting properties are adversely affected⁶. Also when asphaltic bitumens are cutback with creosote oil, its wetting properties are increased. This explains the improved adhesion in the case of a tar-bitumen mixture. Also, low aromatic tars give a better initial bond than a high aromatic tar. In general tar binders show better adhesion properties than bitumens.

To a certain extent the source of the crude in the case of asphaltic bitumen might influence its adhesion to a stone, for, the percentage of asphaltogenic acids can vary from 0 per cent to 7 per cent depending on the source.

4.2.2. Viscosity

Resistance to stripping depends on the consistency of the material and it is said that cracked materials are more resistant than uncracked ones²¹. Also, viscosity of the material used influences the adhesive bond. A low viscosity of the binder means a better initial coating, whereas a high viscosity of the binder (after complete coating) means a greater resistance to its displacement by water. Further, undue increase in viscosity may mean indirectly an increase in surface tension. A thin film of a binder produces the most resistant bond, but too thin a film of a highly viscous binder might crack easily. From all these reasons it can be seen that an ideal binder should display a low viscosity at the time of application but should set up to a high viscosity in as short a time as possible after completely coating all the stone aggregates. A cutback grade using a high viscous binder seems to be a satisfactory material.

One way by which viscosity is increased⁶ is by incorporating a filler after the stone is coated. And if the fillers used give an alkaline reaction (such as using a limestone filler for a paving mixture with tar) resistance to displacement will be more. But it must be noted that the filler should be added subsequently. If the filler is only clayey and it is already present in the form of a thin coat over the stone, the binder only coats the dust and not the stone with a resultant stripping by the action of water subsequently. Some natural asphalts which contain an appreciable amount of undesirable material such as silicated clay, though they may have a high viscosity, behave badly and strip early in the presence of water.

5. METHODS FOR IMPROVING ADHESION

In order to improve the adhesive bond between the stone and the binder and to reduce the tendency for displacement by water, certain surface active agents can be introduced. These 'dopes' are antistripping agents and can act in one of many ways. One way by which their presence helps is by the decrease in the value of the adhesion tension ($\gamma \cos \alpha$) either by decreasing the interfacial tension γ and/or changing the value of the contact angle. Another way is to provide a rigid interfacial film which prevents the establishment of the equilibrium contact angle. Also, the

stone could be made water repellant. A number of materials have been tried such as heavy metal soaps, quaternary ammonium salts, certain organic amines, etc., which when added to the binder reduce the interfacial tension. In the case of wet aggregates with tar, treatment with lime is quite well known. In this case, by the reaction of the alkali and the acid constituents in the tar, the interfacial tension is reduced and the adhesion is improved. Since there is no acid constituent in asphaltic bitumens, when lime is used to coat the wet aggregates, for reaction to take place a metal soap or sulphonated castor oil must be used in conjunction. Cetyl pyridinium bromide and other similar cationic detergents when dissolved in water are adsorbed strongly on to the stone surface thereby resisting stripping. If this adsorbing reagent is applied in solution in creosote oil, an oil film is carried with the reagent on the stone surface and a waterproofing of the stone results.

Thus the three ways by which the adhesion could be improved are:

- (a) to dissolve the agent in the binder,
- (b) to add lime or other alkaline material to the wet stones prior to coating with tar, and
- (c) to treat the stone with water repellants.

The first method is not usually successful since the binder has to be heated to high temperature, when the effect of the adhesive agent is impaired. Best results, however, will be obtained when the agents are used with cold and wet aggregates before the binder is added.

6. CONCLUSION

From the foregoing information it can be seen that the phenomenon of adhesion is very important in bituminous road construction, especially in the case of surface dressings and open premixed carpets or macadams. In all the cases the aggregates must be clean, dry and free of dust. Secondly, cubic and irregular chippings are to be preferred. The binder should have a low viscosity initially and should be capable of building up a high viscosity subsequently. The binder chosen should have as low viscosity as possible to coat and wet the aggregates and it should be consistent with the temperature and climatic conditions of laying such as low viscous binders at low temperatures, etc. As far as possible, the construction should be done during dry weather. During damp and cold weather, the construction should be done more carefully, keeping the drying and volatile properties of the

binder in view. Surface dressing should always be protected and watched in the early days when it is fresh since both traffic and rain immediately after laying are injurious to it. If the aggregates are wet, some of the adhesive agents could very usefully be employed. As a matter of fact in certain European countries addition of agents has become a rule. In the case of open bituminous macadams or pre-mix open carpets also the above mentioned observations hold good. In addition, proper camber should be given to these surfaces so that water may not be entrapped in them for a considerable period of time. Some fillers especially alkaline ones are quite useful if they are incorporated in small quantities. Undue amount of dust in the aggregates prior to coating is very harmful especially if the stones are damp. In such cases priming of the stones with a low viscous oil, or treatment with adhesive agents, would be quite advantageous.

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REFERENCES

1. Sæther, E., "Adhesion between Bitumen and Mineral Aggregates," Statens Vegvesen Veg laboratoriet, Oslo, Norway, Highway Research Abstracts, Vol. 18, No. 4, pp. 8-9, April 1948.
2. "Mechanism of Absorption of Bituminous Materials by Aggregates", Shell Bitumen Reprint No. 4.
3. Shaw, R.J., "Adsorption of Asphalts to Minerals and Related Phenomena," Proc: Arizona Roads and Streets Conference, University of Arizona, Highway Research Abstracts No. 76, pp. 1-2, January 1941.
4. Thelen, Edmund, "Development of an Asphalt Technology," Proc: Association of Asphalt Paving Technologists, Vol. 23, 1954.
5. "Adhesion,"—Road Tar Bulletin No. 4, 1948, British Road Tar Association.
6. "Missouri Boil Test," Missouri State Highway Department.
7. Riedel & Weber Test—On the Adhesiveness of Bituminous Binder on Aggregates, Asphalt und Teer, 33, 677 (1933).
8. "Static Immersion Stripping Resistance Test," (tentative) Sept. 1954. A.S.T.M. on Bituminous Materials for Highway Construction, Waterproofing and Roofing.
9. Broome, D.C., "The Testing of Bituminous Mixtures," Vol. 2, p. 17, Edward Arnold and Co., London 1949.

10. "Stripping of Bitumen from Aggregate," Use of Plate Test, Main Roads, Vol. 20, No. 4, June 1955.
11. Tyler, Owen, R., "Adhesion of Bituminous Films to Aggregates," Purdue University Research Bulletin, No. 62, Vol. 22, Sept. 1938.
12. Pauls, J.T. & Goode, J.F., "Application and Present Status of the Immersion Compression Test," Proc : Association of Asphalt Paving Technologists, Vol. 16, Feb. 1947.
13. Enustun, B.S., "Photo-Electric Estimation of Binder/Aggregate Stripping," Highways & Bridges, 22, (1064) 8, 1954.
14. "Preliminary Report on the Use of Radioactive Isotopes in the Laboratory Determination of Asphalt Stripping from Paving Stones", South Eastern Road Builder, Vol. 1, No. 6, 1951.
15. "The Immersion Wheel Tracking Test," RN/2182/JHN/DHM, Road Research Notes No. 37, Sept. 54.
16. Hallberg, Sten, "The Adhesion of Bituminous Binders and Aggregates in the Presence of Water," Statens Vaginstitut, Stockholm Meddelande, 78, 1950.
17. Gagle Ray & Rivenes Thor, "Adhesion of Asphalt to Aggregates," Centre Line, Vol. 2, No. 4, June 1939.
18. Knight B.H., "Adhesion of Bituminous Binders to Road Aggregates," Surveyor (England), Vol. 93, No. 2400, Jan. 21, 1938.
19. Oberbach, Dr. J., "The Boiling Test for Adhesion after Riedel and Weber and in Practice," Teer Und Bitumen, Vol. 34, No. 24, 20, 1936 & No. 25, September 1936.
20. Pfeiffer, J.P.H., "The Properties of Asphaltic Bitumen," 1950, Elsevier Publishing Company, Inc., New York 1950.
21. Kelley E. F., "Report of Committee on Characteristics of Asphalts," Proceedings, Highway Research Board, Vol. 17, pp. 328-333, 1937.

RESEARCH SECTION

**“ STUDY OF SOFT AGGREGATES FROM
DIFFERENT PARTS OF INDIA WITH A VIEW TO
THEIR USE IN ROAD CONSTRUCTION,
II. LATERITES ”**

By

R. L. NANDA & R. KRISHNAMACHARI*

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SYNOPSIS

This note presents a sample survey of laterite from various parts of India as a highway material which has been used to a limited extent in some parts of the country.

Mechanical properties of laterites as road aggregate have been studied and the data collected has been compared with the more commonly used type of aggregates.

The physico-chemical properties studied in this investigation together with the field performance data collected reveal that laterites are suitable for sub-base and base course construction.

1. INTRODUCTION

This study is in continuation of “ Study of soft aggregates from different parts of India with a view to their use in low cost road construction—I-Rajasthan”, Journal of the Indian Roads Congress, Vol. XXI-3, April 1957, pp. 433-457, in which utilisation of locally available material in Rajasthan had been discussed. In the present investigation, “Laterite as a road aggregate” has been studied.

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This general survey has been undertaken in order to assess the suitability of this soft rock as a road building material. In some parts of India, laterite is the only natural aggregate available and the most common problem which confronts the highway engineer who has to build roads in such regions is that of using local material in order to construct a low cost road. But the engineering properties of this material were not clearly understood so far.

The object of this investigation is, therefore, to present the nature and properties of laterite so that it may be used in areas where harder type of rock is not available within economic distance.

Many thousands of miles of rural or feeder roads will be constructed during the Five-Year Plan to connect agricultural villages with the agricultural markets and the country cannot afford long milages of expensive types of pavements. In order to construct low cost roads locally available materials have to be used to the greatest possible extent.

Laterite is a form of regolith highly peculiar to India and a few other tropical and semitropical countries. In some parts of this country laterite is the only road building material available and hard stone metal is either scarce or is available at high cost. Hence to assist the highway engineer in his efforts to reduce the cost of construction, research on locally available materials hitherto considered unsuitable, has been undertaken.

2. COLLECTION AND SELECTION OF LATERITE SAMPLES

A number of samples were collected from all over India through the Ministry of Transport and various Public Works Departments in the country.

Out of the samples received only 38 could be selected for inclusion in this investigation as others were either too small for all the tests to be performed or more than one sample were received from the same quarry. The particulars of these samples which have been studied are described in Table 1.

3. EXPERIMENTAL DATA

3.1. General

The wide distribution of laterite as seen in Fig. 1, can make it a widely used material for road construction but the engineer-

ring properties of this material have to be clearly understood. In order to assess its suitability for the construction of roads detailed physical and mechanical properties and chemical analysis have been studied.

3.2. Physical and Mechanical Characteristics

The dry bulk density, water absorption and percentage voids were determined on three inch cubes cut from the laterite boulders. The cubes of three inch size were cut by the stone cutting machine as shown in Photo 1. The photographs of some of the laterites are also given in Photos 2 to 5.

3.2.1. Bulk density: The bulk density of the cubes was calculated on their oven dry weight and the samples were dried for constant weight in the oven at 105-110 degrees centigrade. The volumes were taken by direct measurements as per A.S.T.M. C-134-41.

$$\text{Bulk density} = \text{weight gm./volume cc} \times 62.4 = \text{lb per cu. ft.}$$

3.2.2. Specific gravity: The specific gravity of the powdered samples (passing A.S.T.M. sieve No. 100) was determined as per A.S.T.M. C-135-47 with the following exception. Distilled kerosine (fraction collected from 200-240 degrees centigrade) was used instead of distilled water.

3.2.3. Water absorption

(a) **Water absorption of cubes:** The water absorption of oven dried laterite cubes has been evaluated as per A.S.T.M. C-97-47 except for the increase of period from 48 hours to 66 hours.

(b) **Water absorption of aggregates:** The aggregates of $1\frac{1}{2}$ —1 inch size and $\frac{3}{4}$ — $\frac{1}{2}$ inch size were oven dried at 105—110 degrees centigrade for constant weight, cooled to room temperature and weighed (Wt. A). The dried samples were placed in water in suitable vessels and kept immersed, properly removing the air bubbles, for 66 hours and two months. After the above mentioned immersion period, the samples were taken from the water and brought to surface dry condition. They were immediately weighed (Wt. B) and then dried in the oven to constant weight at 105—110 degrees centigrade (Wt. C). Percentage water absorption = $\frac{B-C}{C} \times 100$

Note: (i) The dry weight C is taken instead of A as some of the soft material of laterites goes into water as suspension during immersion.



Fig. 1. Map of India showing location of laterite samples tested

- (ii) The results of this test have some dependence on the size of particles tested, and for this reason samples of burnt brick and quartzite are also taken of the same size.

3.2.4. **Voids:** The percentage voids were calculated as follows:

$$(SG \times 62.4 - Db / SG \times 62.4) \times 100$$

SG = specific gravity.

Db = bulk density.

As the aggregates in a road are subjected to the destructive influence of impact, crushing and abrasive forces, their ability to withstand these forces depends largely on the mechanical properties of the material concerned. Hence these properties are studied in detail. The compressive strength of the samples was determined on three inch cubes whereas impact, crushing value and abrasion values were determined as per standard methods given below under the respective tests.

Laterite is a soft material in comparison to conventional road stone and it is desirable to find out its behaviour under impact, crushing and abrasion when it is wet. The test specimens were soaked in tap water for 66 hours before they were tested.

3.2.5. Compressive strength

(a) **Compressive strength test (dry):** Samples of three inch cubes were cut on stone cutter. In the case of brick the frogs were filled in by neat cement paste and allowed to age for 24 hours and then capped with plaster of Paris. The capped specimens were tested after 24 hours at a rate of 100 lb per sq. in. per second.

(b) **Compressive strength test (wet):** The three inch cubes were immersed in water for 66 hours. At the expiry of this period the specimens were taken out and brought to surface dry condition. The testing procedure was followed as in the dry state.

3.2.6. Impact test

(a) **Impact test (dry):** The size of the test sample was $\frac{1}{2}$ – $\frac{3}{4}$ inch. The procedure followed was as per B.S. 812-1951. Aggregate under test is shown in Photo 6,

(b) **Impact test (wet)**: The sample was prepared, oven dried, measured and weighed quantity taken as per B.S. 812-1951. The test sample was then immersed in water for 66 hours. At the end of this immersion period, it was taken out and brought to surface dry condition and tested as per standard method. The material was sieved through B.S. sieve No. 7 and the fraction retained was washed, oven dried to constant weight and weighed. This weight was subtracted from the test sample, which represented the fraction passing B.S. sieve No. 7. The results were calculated as per standard method.

3.2.7. Aggregate crushing value

(a) **Aggregate crushing value (dry)**: The test was performed on $\frac{1}{2}$ - $\frac{3}{8}$ inch size aggregate as per B.S. 812-1951.

(b) **Aggregate crushing value (wet)**: The sample was prepared, oven dried, measured and weighed quantity taken as per standard method. The wet sample was tested and results calculated as described under wet impact test.

3.2.8. Los Angeles abrasion value

(a) **Los Angeles abrasion value (dry)**: For the Los Angeles abrasion test the procedure given as per A.S.T.M. C 131-51 was followed using aggregates of $\frac{3}{4}$ - $\frac{1}{2}$ inch and $\frac{1}{2}$ - $\frac{3}{8}$ inch sizes in equal proportions.

Note: This size range is nearer to the size adopted in impact and crushing value test.

(b) **Los Angeles abrasion value (wet)**: The test sample was taken as per standard method. It was immersed in water for 66 hours. After this immersion period the sample was taken out, surface dried, tested and results calculated as per standard procedure.

3.3. Results of Physical & Mechanical Tests

The results of bulk densities, specific gravity, water absorption (cubes) and voids are given in Table 2 whereas the results of water absorption of aggregates are shown in Table 3. The results of other mechanical tests both dry and wet are given in Tables 4 and 5.

3.4. Chemical Analysis

Representative samples of laterites were taken, crushed and ground to pass No. 100 (A.S.T.M. sieve). They were dried in the oven at 105-110 degrees centigrade. The percentage oxide

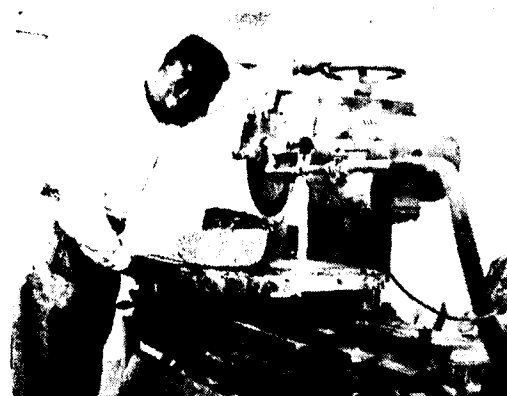


Photo 1.
Sample of laterite
cube being cut in a
stone cutting
machine

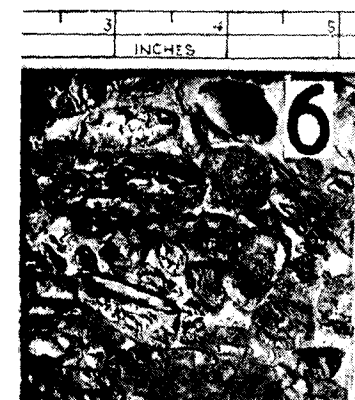


Photo 2.
A sectional view of laterite
from Assam. Serial No. 6

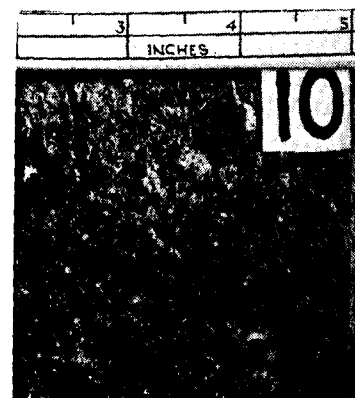


Photo 3.
A sectional view of laterite
from Bihar.
Serial No. 10

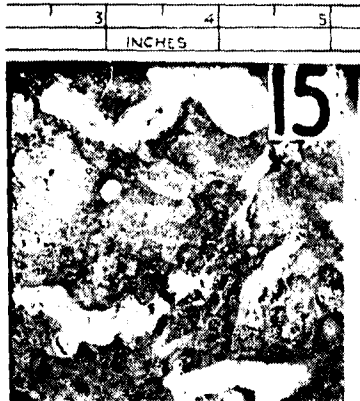


Photo 4.
A sectional view of laterite
from Kolhapur, Bombay.
Serial No. 15

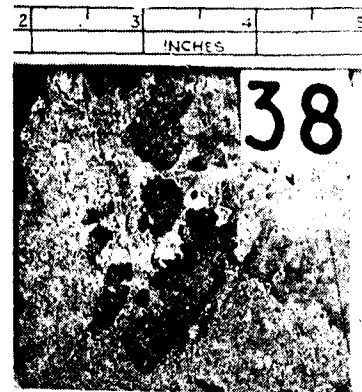


Photo 5.
A sectional view of laterite
from Mirzapur, Uttar Pradesh.
Serial No. 38



Photo 6.
Laterite aggregate being tested
for aggregate impact value

composition was determined on the basis of moisture free samples. The procedure for chemical analysis was adopted as per standard methods of A.S.T.M. C 18-52 except for Fe_2O_3 where C 114-47 was followed.

The results of the chemical analysis are given in Table 6.

4. EVALUATION OF LATERITE

There are no fixed standards in India to the mechanical properties of road stones. The specifications framed by A.S.T.M., A.A.S.H.O. and the limits suggested by Road Research Laboratory, Harmondsworth, England, cannot be followed rigidly in this country as the traffic and climatic conditions here are quite different from those existing in those countries. In the absence of such specified limits the mechanical properties of laterite have been compared with those of burnt brick (Delhi 1st class) which is generally used as soling material and has successfully stood the test of time. The results are also compared with quartzite (Delhi) which is an established sound road making material.

From the results of various tests the statistical averages with their errors have been calculated. They are $A.M. \pm \sigma$

$A.M.$ = Arithmetic mean.

σ = Standard deviation.

The results of laterite in comparison to burnt brick and quartzite are given in Tables 7, 8, 9 and 10.

5. DISCUSSION OF TEST DATA

5.1. Physical and Mechanical Characteristics

Correlation analysis has been carried out for the physical and mechanical test values and the results are given in Table 12. The coefficient of correlation values given in the Table shows the degree of relationship between the two variables. This value ranges from -1 to $+1$. A positive value of the coefficient of correlation indicates that generally the higher values of the one variable are associated with the higher values of the other variable and vice versa. The negative value indicates that higher values of one variable are associated with the lower values of the other variable and vice versa. The significance of the coefficient of correlation values has also been tested and, in general, a value of 0.5 or more can be taken as significant for this Table.

5.1.1. **Dry bulk density:** The dry bulk density of laterite varies from 114 to 183 lb per cu. ft. as shown in Table 2. The average bulk density of this material is 135.1 lb per cu. ft., based on 38 samples. The statistical analysis reveals that, if a large number of samples are tested, in two out of three (66.7 per cent) the average value (true average) will lie between 133.1 to 145.9 lb per cu. ft.

The bulk density of burnt brick on the other hand is only 103.5 lb per cu. ft. Thus laterite as a whole has greater compactness than burnt brick.

The dry bulk density of the laterite works out to about 85 per cent of that of quartzite.

5.1.2. **Voids:** The voids of laterite vary from 5 to 59 per cent. The average void content is 24.1 per cent as is shown in Table 2.

This compares reasonably well with burnt brick which has 34 per cent voids.

5.1.3. **Specific gravity:** Though no standard specification limits are available regarding the value of the specific gravity of road aggregates, this test is carried out on laterite samples with a view to study if there is any significant correlation between this test value and the structural behaviour of the laterite. But the data obtained as specific gravity of the various samples indicates that it is not possible to predict the quality of the material by specific gravity test alone. This value of specific gravity is also used for determining the percentage voids after knowing the dry bulk density. The specific gravity of laterite is varying from 2.2 to 4.6. The average specific gravity comes out to 2.97 whereas of burnt brick it is only 2.51.

The large deviation in laterite may be due to the presence of different mineral constituents in different proportions.

5.1.4. **Water absorption:** The average value of water absorption in 3 inch cubes, $1\frac{1}{2}$ -1 inch and $\frac{1}{2}$ - $\frac{3}{8}$ inch aggregates at the end of 66 hours is 9.5, 9.8 and 9.7 per cent respectively as is shown in Tables 2 and 3.

The average water absorption capacity of laterite is less than that of burnt brick which has as high values as 13.2, 17.0 and 19.5 per cent for 3 inch cube, $1\frac{1}{2}$ -1 inch and $\frac{1}{2}$ - $\frac{3}{8}$ inch sizes respectively, as described in Tables 3 and 10.

In case of laterite aggregates the increase in water absorption from 66 hours to two months varies from 0.1 to 1.6 per cent and 1.5 to 3.4 per cent in $\frac{1}{2}$ - $\frac{3}{8}$ inch and $1\frac{1}{2}$ -1 inch sizes respectively. But the corresponding increase in burnt brick aggregate is 3.3 and 6.1 per cent as is shown in Table 3.

Taking into consideration the porosity of laterite it is clear that all the voids are not filled with water even after prolonged immersion of 66 hours or even two months. This is probably due to the non-continuity of porosity in laterite.

5.1.5. **Compressive strength:** The compressive strength of three inch cubes of laterite in dry state ranges from 11000 to 600 lb per square inch as is shown in Table 4. The average value is 3385 lb per square inch and the true average lies in the range of 5350-2725 lb per square inch with 66 $\frac{2}{3}$ per cent probability.

But its compressive strength in wet condition varies from 10500 to 100 lb per square inch. The average works out to 2168 lb per square inch and the true average lies in the range of 3000-1335 lb per square inch, with 66 $\frac{2}{3}$ per cent probability.

Most of these values of laterite compare favourably with burnt brick both in dry and wet state as the average values of burnt brick are 1356 and 1249 lb per square inch respectively.

Bricks usually display wide scattering of results in their crushing strength but the variation in results exhibited by laterites is far more both within and between different types of laterites. This is probably due to the different type of vesicular nature of the material and variation in their chemical constituents.

Moreover, the results of the tests carried out on aggregates are much more reproducible than the tests made on single pieces of rock. Road Research Laboratory, Harmondsworth, has found that one Los Angeles abrasion test or aggregate crushing value test gives the same probability of a true result as 60 crushing strength tests. This is due to the fact that variations between individual pieces of stone tend to cancel each other in tests on aggregates.

This fact has also been proved in this investigation by the study of coefficient of variations, as given in Table 11.

Thus the results obtained under compressive strength tests are not reproducible and uncertain and for this reason while evaluating the test results reliance is placed on those tests which are performed on aggregates such as impact, crushing and abrasion rather than compressive strength of rock pieces.

5.1.6. **Aggregate crushing value:** The test values (dry state) vary from 20 to 60 per cent as shown in Table 4. The average value is 37.3 per cent and the true average lies between 36.8 and 38 per cent with 66 $\frac{2}{3}$ per cent probability.

In the absence of any standard limits for crushing value of the materials to be used in the base course as well as surface course, the results of laterite are compared with those of burnt brick (Delhi 1st class) and quartzite (Delhi).

The tentative limits suggested by Road Research Laboratory, England, for crushing value of the material to be used in water-bound macadam are 30, 23, and 13 per cent, in the case of light, medium and heavy traffic respectively.

Even though the crushing value of laterite does not compare favourably with quartzite having 20 per cent of crushing value, it compares favourably so with that of burnt brick. The average crushing value of burnt brick (Delhi 1st class) is 59 per cent whereas the average value of laterite is 37.4 per cent. From the samples tested it is clear that 95 per cent of the samples have lesser crushing value than brick and are therefore superior to brick. Hence laterite can safely be used for sub-base and base course construction.

5.1.7. **Aggregate impact value:** The test values vary from 16 per cent to 77 per cent as given in Table 4. The average value works out to be 37.7 per cent and the true average lies between 36.4 and 37.6 per cent with 66 $\frac{2}{3}$ per cent probability.

But as the results of aggregate crushing value and aggregate impact value are very close to each other, without any significant difference, for purposes of evaluation the limits of impact value are taken just as in the case of aggregate crushing value. The results in Table 12 indicate that there is good correlation between the two tests which works out to be $+0.92$.

The results of this test also confirm the same deduction as in the case of aggregate crushing value, that is, laterite is stronger than burnt brick and can safely be used in sub-base and base course.

5.1.8. **Los Angeles abrasion value:** The Los Angeles abrasion value of laterite varies from 17 to 97 per cent as given in Table 4. The standard specifications for crushed stone, crushed slag and gravel for water-bound macadam base and surface courses of pavements as per A.S.T.M. designation D 694—49 are 60 and 40 per cent maximum respectively.

Out of laterite samples tested about 80 per cent will satisfy the A.S.T.M. standard for base course. The results of Tables 4 and 10 indicate that based on Los Angeles abrasion value laterite is superior to brick.

5.1.9. **Mechanical properties of wet aggregates:** The laterite gets partly softened on wetting and loses some of its strength. The average impact and crushing value of laterite in wet state is 45.5 per cent and 47.5 per cent respectively. Whereas in case of burnt brick the average impact and crushing value in wet state is of the order of 72 and 65 per cent respectively, as shown in Table 5.

From this study it is clear that even wet laterite is in no way inferior to wet burnt brick which is a sound base course material and has stood the test of time. Moreover, its complete wetting in actual base course construction does not arise except in some waterlogged areas.

The results of wet laterites in Los Angeles abrasion test are lesser than oven dry state. This is contrary to the results obtained with wet aggregates in impact and crushing value tests.

Probably this is due to the fact that wet laterites get stuck to the wall of the drum and there is not much abrasion between aggregates as well as between metallic balls and aggregates. Moreover, there is less crushing of wet aggregates due to the impact of falling balls because of wet fines acting as cushion. Hence this test is unreliable for laterites in wet condition.

5.2. Chemical Analysis

Chemically a laterite is essentially composed of a mixture of hydrated oxides of aluminium and iron of variable proportions, often with a small percentage of other oxides.

The hardening of laterite is thought to be due to:

- (i) Dehydration of the hydroxides of iron and aluminium present.
- (ii) Further oxidation of the high iron content of the material.
- (iii) The precipitation of these oxides as cementing material.

But from the experimental data collected there seems to be no significant correlation between the chemical constituents and mechanical properties.

This is probably due to the heterogeneous nature of laterite and variation in porosity from sample to sample.

6. FIELD PERFORMANCE OR SERVICE BEHAVIOUR

In order to study the correlation of the mechanical properties with service behaviour, a questionnaire was sent to collect the information on field performance of the laterite covered in this investigation. The questionnaire is given in Appendix 1.

The laterites are classified as satisfactory and unsatisfactory according to their service behaviour. The limited information obtained from various highway officials is summarised in Table 13.

With the available information all the samples having impact, crushing and abrasion values upto 46, 47, and 46 per cent respectively are behaving satisfactorily as base course. But so far as wearing course is concerned they are not giving good service especially under heavy iron-tired traffic due to the formation of ruts in rainy season.

7. CONCLUSIONS

The results of this study indicate the following:

7.1. The majority of laterites have sufficient mechanical strength and resistance and are stronger than burnt brick which is widely used for soling. Thus most of the laterites tested with their aggregate impact values ranging between 20 per cent to 50 per cent are suitable for subbase and base course construction wherever available economically.

7.2. The laterite gets partly softened when wet and loses some of its mechanical strength and wears quickly and hence is not so suitable for wearing course construction, while being suitable for subbase and base courses.

7.3. There is no significant correlation between the chemical constituents and mechanical properties of laterite.

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BIBLIOGRAPHY

1. A.A.S.H.O., "Standard Specifications for Highway Materials, and Methods of Sampling and Testing." Parts I and II, 1950.
2. American Society for Testing Materials, A.S.T.M. Part III, 1952.
3. C.R.R.I., "An Investigation of Some Laterites," Road Research Notes, No. 4, 1953.
4. Bauer, E.E., "Highway Materials," 1932.
5. Bedrich Fruhauff "A study of Laterite Soils" Proceedings of the Highway Research Board, U.S.A., Vol. 26, 1946, pp. 579-593.
6. British Standard Institution, B.S. 812-1951 "Sampling and Testing of Mineral Aggregates and Sand Fillers," London, 1951.
7. "Experimental Surface Dressing over Laterite Macadam on the Baripada-Balasore section of Madras-Calcutta Road"—Transport Communications Monthly Review of Indian Roads Congress, No. 91, November 1954.
8. Fermor, L.L., "What is Laterite"—Geological Magazine, London, Decade V, Vol. VIII, 1911, pages 454-462, 507-516 & 559-566.
9. Final Report of the All India Soil Survey Scheme, Bulletin of the Indian Council of Agricultural Research, No. 73, 1953.
10. Fox, C.S., "The Bauxite and Aluminous Laterite Occurances of India". Geological Survey of India 1936.
11. Knight, B.D., "Laterite as a Road Making Material," Contractors Record and Public Works Engineer, Vol. 5, No. 1, March 1953, pp. 31-32.
12. Knight, B.H. "Road Aggregates, their Uses and Testing," Edward Arnold, 2nd Edition, 1948, Vol. 3, pp. 102-103, 108, 116, 126-128, 151-152, 156-159 & 188-223.
13. "Laterite and Laterite Soils"—Imperial Bureau of Soil Science Technical Communication No. 24, 1932, London.
14. Lewis, D.W. & Woods, K.B., "Research as related to Aggregate specifications", Proceedings 35th Purdue University, 1949, pp. 155-173.
15. Mehra, S.R., "Soil Stabilisation with Soft Aggregates", Proceedings 2nd International Conference on Soil Mechanics, Vol. 4, 1948, pp. 252-257.
16. Pendleton, R.L., & Sharaswane, "Analysis of Siamese Laterite", Soil Science, 62, 46, pp. 423-440.
17. Phemister, J. & Markwick, A.H.D., "Road Stone Geological Aspects and Physical Tests", Road Research Special Report No. 3, 1946, pp. 25-38.

18. Prescott, J.A. & Pendleton, R.L., "Laterite and Lateritic Soils". Commonwealth Bureau of Soil Science, Technical Communication No. 47, 1952, page 9.
19. Report on the Dacca University Scheme on "Laterite and Red Soils of India"—1930—1941. Published by Indian Council of Agricultural Research, New Delhi, India.
20. Shergold, F.A., "A Review of available Information on the Significance of Road Stone Tests", Road Research Technical Paper No.10, 1948, pp. 2,4,11.
21. Shergold, F.A. & Lewis, F.V., "Study of Aggregate Impact Test", Road Research Lab., RN-1098-F.A.S. F.V.L., Feb. 1949.
22. Sweet, H.S., "Physical and Chemical Tests of Mineral Aggregates and their Significance," Symposium on Mineral Aggregates, A.S.T.M., 1948, pp. 49—69.
23. Wadia, D.N., "Geology of India," Macmillan & Co., London, III Edition, 1953, pp. 52, 109, 302, 398—402, 471 & 543.
24. Wills, E.A. & Kelly, J.A., "Mineral Aggregates for Low Cost Roads and Water-Bound Macadams" Symposium on Mineral Aggregates, A.S.T.M. 1948, pp. 183—194.
25. Uppal, H.L. and Bhupindar Singh, "Study of Soft Aggregates from Different Parts of India with a view to their use in Low Cost Road Construction—I, Rajasthan, Journal of the Indian Roads Congress, Vol. XXI—3, 1957, pp. 433—457.
26. Winterkorn, H.F. and Chandrasekharan, E.C. "Laterite Soils and their Stabilization," Bulletin of the Highway Research Board, U.S.A., No. 44, 1951.

TABLE I
Particulars of laterites

S. No.	State	Location	Name of quarry
1	2	3	4
1.	Andhra	East Godavari District	Kesavaram.
2.	do.	Tadapalligudam, West Godavari District	Arugolanu.
3.	Tripura	Agaratala	—
4.	Assam	Malidhar	Kathirail.
5.	do.		Hilara.
6.	do.	do.	Sunagiri.
7.	do.	Tura-South Garo Hills	—
8.	do.	do.	—
9.	do.	Gauhati—Shillong Road, Shillong	—
10.	Bihar	Jamsola, P. O. Baharagora, Singh- bhum District	—
11.	Bombay		From village Sinod in Tal Mansrole, Distt. Surat, Bombay, (12 miles from Anklesh- var).
12.	do.		Mohar river bed near Kapadvaraj town, Kapadvaraj, Attar- sumba road, M-1.
13.	do.	Ratanagiri	—
14.	do.	Alnavar	—
15.	do.	Kanakavali, Kolhapur	—
16.	Madhya Bharat Mandsaur		Mhow - Neemuch road, M-118-119, Mandsaur.
17.	do.	do.	Gwal Toli Neemuch, Nasirabad, mile-1, Mandsaur.

TABLE 1 (contd.)

S. No.	State	Location	Name of quarry
1	2	3	4
18.	Madhya Bharat	Bhilsa	Bhatni Gunj, Basoda.
19.	do.	Bercha No. 2	—
20.	do.	Bercha No. 1	—
21.	Madhya Pradesh	Balaghot, Nagpur	—
22.	do.	Harda	Bisoni
23.	do.	Seoni	—
24.	do.	Bhandara	—
25.	Madras, East Coast	South Arcot Distt.	Seplanatham.
26.	Madras, West Coast	—	—
27.	do.	—	—
28.	do.	—	Mile 14/6 of Puthana-thani, Trivegapura Road.
29.	do	B-Mangalore, Cherathore Road 8/4 one furlong off.	—
30.	do.	D-Mangalore Cherathore Road 14/6 Road side.	—
31.	do.	Malabar (Kurumbanad Taluk).	On right side of mile 1/7 of Badagara Villiappally Road.
32.	do.	—	Dharmadam quarry at 0/5 Dharmadam Mamamkunnu Road.
33.	do.	—	74/4 of Cherthayan Perumbay Road.
34.	Rajasthan	Jhalawar	—
35.	Travancore-Cochin	Warkala	Vettoor.
36.	Uttar Pradesh	Sample 3	Moradabad.
37.	do.	Roberts Ganj, Mirzapur.	—

TABLE 2
Physical properties of laterite

Sl. No.	State	Location	Bulk density in lb per cu. ft. (average of 3 in. cubes)	Specific gravity passing No. 100 B. S. material	Voids per cent	Absorption after 66 hrs. soaking in water percentage (average of three inch cubes)
1	2	3	4	5	6	7
1.	Andhra	Kesavaram Quarry	177.3	2.83	5.0	4.0
2.	Andhra	Arugolamu Quarry	156.2	2.89	14.0	5.3
3.	Tripura	Agartala	118.1	3.32	42.9	9.2
4.	Assam	Kathirail Malidhar, Shillong	132.9	3.32	35.8	7.4
5.	"	Hilara Malidhar, S. D. Shillong	122.0	2.85	31.3	10.4
6.	"	Sunagiri Malidhar, S.D. Shillong	121.6	3.10	37.1	10.0
7.	"	Tura—South Garo Hills	130.8	2.67	20.8	7.2
8.	"	Tura—South Garo Hills	128.5	2.60	20.8	7.3
9.	"	Gauhati—Shillong Road, Shillong	121.1	3.88	50.0	22.8
10.	Bihar	Jamsola P.O. Baharagora, Singhbhum District	143.6	2.85	19.2	5.2
11.	Bombay	Kharod hard moorum Sinodintal, Surat	135.5	2.39	9.2	11.8
12.	"	Mohar river-bed near Kopadvaraj Dakor	178.8	3.71	22.9	4.1
13.	"	Ratnagiri	124.9	2.85	30.4	14.0
14.	"	Alnavar	126.4	2.20	8.4	9.8
15.	"	Kanakavali, Kolhapur	140.7	2.82	18.6	8.6

TABLE 2 (contd.)

16.	Madhya Bharat	Mhow—Neemuch Road, M. 118, Mandsaur	178.3	3.73	23.3	4.3
17.	"	Gwal Toli Q Neemuch, Nasirabad, Mile 1, Mandsaur	182.9	3.59	18.4	4.8
18.	"	Bhatni Q, Gunj Basoda, Bhilsa	116.1	4.60	59.5	16.9
19.	"	No. 2, Bercha	159.1	4.11	37.9	6.6
20.	"	No. 1 Bercha	149.9	3.88	38.1	8.1
21.	M. Pradesh	Balaghot, Nagpur	130.0	2.81	25.9	9.6
22.	"	Bisoni Q, Harda	166.6	2.79	5.0	5.7
23.	"	Seoni	147.6	2.69	12.0	10.0
24.	"	Bhandara	138.8	2.57	13.6	7.8
25.	Madras	Seplenatham	136.6	2.38	8.3	7.6
26.	East Coast	N. A.	143.3	2.48	5.4	6.2
27.	Madras	N. A.	121.3	2.63	25.9	13.2
28.	"	Mile 14/6 of Puthanthani, Trivegapura Road	148.6	2.79	17.7	6.3
29.	"	Mangalore	130.1	2.82	36.8	10.7
30.	"	D. Mangalore (Cherathore road)	125.8	2.75	37.0	12.9
31.	"	Badagara, Malabar	114.9	2.95	37.2	16.0
32.	"	Dharanadam Q	142.5	2.48	9.6	7.1
33.	"	Cherathayan, Perumbay Road	138.4	2.77	19.8	9.0
34.	Rajasthan	Jhalawar	161.1	3.10	16.8	7.1
35.	T. Cochin	Vettoor Q, Warkala	102.4	2.63	39.1	23.6
36.	"	N. A.	114.8	2.27	19.7	14.1
37.	Uttar Pradesh	Sample 3, Moradabad	129.1	2.64	21.7	11.0
38.	"	Roberts Ganj, Mirzapur	165.5	3.34	21.5	6.4
		Average ...	135.1	2.97	24.14	9.5

TABLE 3

Water absorption in aggregates

Range of absorption	Sl. No.	Absorption							
		66 hours				2 months			
		3 in. cubes	1½ in.-1 in. Agg.	¾ in.-½ in. Agg.	3 in. cubes	1½ in.-1 in. Agg.	Variation	¾ in.-½ in. Agg.	Variation
Higher	35	23.6	—	15.8	—	—	—	17.1	+1.3
	9	22.8	13.5	19.3	—	15.5	+2.0	19.9	+0.6
	18	16.9	—	11.1	—	—	—	12.4	+1.3
	31	16.0	15.3	13.5	—	17.6	+2.3	13.8	+0.3
	36	14.1	12.9	12.7	—	16.3	+3.4	12.8	+0.1
	13	14.1	—	10.9	—	—	—	11.6	+0.7
Lower	38	6.4	—	5.4	—	—	—	6.0	+0.6
	2	5.3	6.7	6.6	—	8.7	+2.0	8.2	+1.6
	10	5.2	5.9	5.9	—	7.4	+1.5	6.3	+0.4
	17	4.8	4.3	4.9	—	6.0	+1.7	5.4	+0.5
	16	4.3	—	5.5	—	—	—	6.3	+0.8
	12	4.1	—	4.9	—	—	—	5.7	+0.8
Average :		9.5	9.8	9.7	—	11.9	+2.1	10.4	+0.7
Burnt brick (1st class): Delhi		13.2	17.0	19.5	—	23.1	+6.1	22.8	+3.3
Quartzite Delhi :		Nil	0.4	0.7	—	0.5	+0.1	1.1	+0.4

TABLE 4
Mechanical properties of laterite

Sl. No.	State	Location	Compressive strength in lb. per sq. in. (Average of three inch cube)		Aggregate impact value per cent (Dry)		Aggregate crushing value per cent (Dry)		Los Angeles abrasion value per cent (Dry)	
1	2	3	4 (Dry)	5 (Wet)	6	7	8	9	10	11
1.	Andhra	Kesavaram Quarry	6174	3161	23.7	27.9	34.1	34.1	34.1	34.1
2.	"	Arugoluru Quarry	4183	3061	26.8	34.1	48.8	48.8	48.8	48.8
3.	Tripura	Agartala	927	1445	47.7	48.3	28.4	28.4	28.4	28.4
4.	Assam	Kathirail Malidhar, Shillong	1062	1742	36.6	38.4	33.3	33.3	33.3	33.3
5.	"	Hilara Malidhar, S.D. Shillong	1427	948	31.0	38.8	33.4	33.4	33.4	33.4
6.	"	Sunagiri Malidhar, S.D. Shillong	1286	1087	28.2	31.4	96.5	96.5	96.5	96.5
7.	"	Tura-South Garo Hills	4984	2676	76.8	59.8	96.7	96.7	96.7	96.7
8.	"	Do.	2696	2427	73.6	59.6	45.2	45.2	45.2	45.2
9.	"	Gaubati, Shillong Road, Shillong	2149	871	31.6	32.4	41.8	41.8	41.8	41.8
10.	Bihar	Jainsola P.O., Baharagora, Singhbhum District	3692	2373	30.7	31.0	18.1	18.1	18.1	18.1
11.	Bombay	Kharod Sindhtal, Surat	5257	2937	33.8	28.6	17.3	17.3	17.3	17.3
12.	"	Mohar river-bed near Kopadvaraj, Dakor	11034	10494	43.5	11.1	37.7	37.7	37.7	37.7
13.	"	Ratnagiri	1925	1481	41.8	37.7	43.7	43.7	43.7	43.7
14.	"	Ahavar	1493	415	41.8	37.7	41.0	41.0	41.0	41.0
15.	"	Kanakavali, Kolhapur	4380	1700	35.7	36.5	28.9	28.9	28.9	28.9
16.	Madhya Bharat	Mhow-Neemuch Rd., mile 118, Mandasaur	8238	6346	34.6	38.7	46.0	46.0	46.0	46.0
17.	"	Gwal Toli Q. Neemuch-Nasirabad, M. I., Mandasaur	7591	4314	33.4	32.5	39.0	39.0	39.0	39.0
18.	"	Bhatni Q., Gunj Basoda, Bhilsa	1732	933	37.0	37.6	31.5	31.5	31.5	31.5
19.	"	No. 2, Bercha	3775	1120	37.8	34.2	34.0	34.0	34.0	34.0
20.	"	No. 1 Bercha	6388	3277	19.1	20.4	50.8	50.8	50.8	50.8
21.	Madhya Pradesh	Balaghot, Nagpur	2738	1273	42.4	42.8	44.6	44.6	44.6	44.6
22.	"	Bisoni Q., Harda	2572	4552	33.4	33.7	42.2	42.2	42.2	42.2
23.	"	Seoni	5824	5898	39.7	39.3	41.5	41.5	41.5	41.5
24.	"	Bhandara	3941	1860	40.0	37.4	44.2	44.2	44.2	44.2
25.	Madras East Coast	Seplanatham	1706	739	47.0	46.6	64.2	64.2	64.2	64.2
26.	Madras West Coast	—	1941	1261	24.2	32.9	37.0	37.0	37.0	37.0
27.	"	—	1444	415	45.7	46.8	—	—	—	—
28.	"	Milo 14/6 of Puthanthani, Trivegapura	2506	1634	26.1	31.2	31.5	31.5	31.5	31.5
29.	"	Mangalore	1361	979	23.2	28.5	24.2	24.2	24.2	24.2
30.	"	D. Mangalore, Cherathore Road	1128	569	50.8	51.9	62.1	62.1	62.1	62.1
31.	"	Badagara, Malabar	763	207	45.5	46.1	69.8	69.8	69.8	69.8
32.	"	Dharamandam Q.	2912	2249	24.1	30.5	25.6	25.6	25.6	25.6
33.	"	Cherathore, Perumbay Road	3368	1734	21.5	29.2	24.1	24.1	24.1	24.1
34.	Rajasthan	Jhalawar	5167	2483	28.6	32.2	—	—	—	—
35.	T. Cochín	Vettoor Q., Warkala	581	108	64.4	47.2	79.9	79.9	79.9	79.9
36.	"	Sample 3, Muradabad	738	232	37.3	36.8	58.8	58.8	58.8	58.8
37.	Uttar Pradesh	Roberts Garj, Mirzapur	1480	531	33.5	36.4	40.8	40.8	40.8	40.8
38.	"	AVERAGE:	8089	2862	15.9	24.1	19.3	19.3	19.3	19.3
			3385	2168	37.7	37.3	43.1	43.1	43.1	43.1

TABLE 5

Mechanical properties of wet aggregates

Sl. No.	Moisture absorption range	Per cent water absorption 66 hours	Impact value per cent	Crushing value per cent	Los Angeles abrasion value per cent
35	Higher	15.8	66.2	71.5	—
9		19.3	57.5	54.9	43.8
18		11.1	38.9	43.4	—
31		13.5	66.3	64.1	58.1
36		12.7	61.9	60.6	52.4
13		10.9	48.9	48.0	—
38	Lower	5.4	28.4	35.8	—
2		6.6	37.3	47.0	28.1
10		5.9	39.4	36.2	42.1
17		4.9	37.0	40.0	28.3
16		5.5	42.1	41.2	—
12		4.9	22.0	27.6	—
Average :			45.5	47.5	
Burnt brick (1st class) :			72.0	65.0	

TABLE 6
Chemical analysis of laterite

Sl. No.	State	Chemical analysis percentage					Mol - Ratio			Morphological characteristics.
		SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	Loss on ignition	SiO ₂		Al ₂ O ₃		
						Al ₂ O ₃ ± Fe ₂ O ₃	SiO ₂			
1	2	3	4	5	6	7	8	9		
1.	Andhra	22.64	31.9	33.4	9.74	0.07	11.2		Brownish red, tinge of yellow, sl. honeycombed.	
3.	Tripura	34.38	8.30	47.20	9.32	1.48	7.04		Dark red with big shells, some of them filled with white material.	
7.	Assam	81.04	2.40	14.00	1.78	3.58	57.26		Blackish red coarse stone with quartz grains looking like sand stone.	
9.	Assam	5.0	27.3	62.2	6.0	0.12	0.31		Brick red clayey some portions blackish red too much colloidal matter.	
10.	Bihar	60.18	10.1	24.8	5.6	3.85	10.1		Dark red with a tinge of light grey no shell.	
12.	Bombay	2.98	27.45	55.2	14.13	0.08	0.18		Reddish brown, solid, some black spots.	
16.	Madhya Bharat	14.28	12.75	59.2	12.5	0.47	1.9		Yellow to brown without any shell.	
21.	Madhya Pradesh	43.98	22.15	25.00	8.53	1.93	3.36		Colour light brown to black brown, no shell.	
22.	do.	52.28	5.90	31.6	7.42	3.40	15.02		Heterogenous coloured, dark brown, red, yellow, some quartz visible.	
29.	West Coast	17.28	18.4	49.6	15.75	0.59	1.59		White, brown & dirty red (honeycombed mass).	
34.	Rajasthan	13.32	26.6	48.2	10.93	0.39	0.85		Dark brown to pink colour, no shells.	
36.	T. Cochin	49.18	27.2	14.80	8.81	2.26	3.07		Red solid mass.	
									Weathered - clayey—with patches of loose greyish material, brick red in colour.	
38.	Uttar Pradesh	16.52	14.60	59.80	8.11	0.52	1.92		Very dense and homogenous, very fine material. Blackish.	

TABLE 7

Comparison of laterite with burnt brick

Mechanical tests	Values for brick in per cent	Per cent samples of laterite	
		Inferior than brick	Superior than brick
Impact value	59	5	95
Crushing value	59	5	95
Los Angeles abrasion value	99	Nil	100

TABLE 8

Comparison of laterite with quartzite (Delhi)

(ABSTRACT)

Mechanical tests	Values for quartzite in per cent	Per cent samples of laterite	
		Inferior than quartzite	Superior than quartzite
Impact value	21	95	5
Crushing value	20	100	Nil
Los Angeles abrasion value	25	86	14

TABLE 9

Comparison of laterite with quartzite

(IN DETAIL)

Mechanical test	Values for quartzite in per cent	Percentage laterite strength in comparison to the strength of quartzite†	Percentage of the total laterite samples
Impact value	21	84	24
		60	40
		47	21
		38	5
		32	—
Crushing value	20	28	5
		80	18
		57	55
		44	16
		36	5
Los Angeles abrasion value	25	31	5
		27	—
		83	28
		62	30
		50	8
		42	8
		36	3
		31	3
		27	6

† Taking the strength of a quartzite as 100 per cent.

TABLE 10

**Comparative mechanical properties of laterite,
burnt brick and quartzite**

Mechanical properties	Laterite (representa- tive range)	Burnt brick (1st class)	Quartzite
1. Bulk density lb per per cu. ft.	133—146	103	163
2. Water absorption per cent	8—11	13	Nil
3. Compressive strength (dry) lb per per sq. in.	2725—5350	1356	28970
4. Compressive strength (wet) lb per per sq. in.	1350—3000	1249	28970
5. Aggregate impact value per cent	36—38	59	21
6. Aggregate crushing value per cent	37—38	59	20
7. Los Angeles abrasion value per cent	24—67	99	42

TABLE 11

**Coefficient of variation and reproducibility of
mechanical tests**

Test	Mean	Coeffi- cient of varia- tion	No. of repeti- tions required	With 0.9 probability expected variation of the mean from the true mean per cent
1	2	3	4	5
1. Bulk density	135.1	4.6	6	± 3
2. Water absorption per cent	9.5	17.2	3	± 16
			8*	± 10
			32*	± 5
			89*	± 3
3. Compressive strength (dry)	3385	32.4	3	± 31
			28*	± 10
			113*	± 5
			314*	± 3
4. Compressive strength (wet)	2168	38.5	3	± 36
			40*	± 10
			160*	± 5
			445*	± 3
5. Aggregate impact value	37.7	1.7	2	± 2
6. Aggregate crushing value	37.3	1.5	2	± 2

Note: Items marked '*' are the calculated replications required to obtain the average value with the variations marked against each in column 4 at 90 per cent probability.

TABLE 12

Coefficient of correlation between mechanical tests

Sl. No.	Variates	Coefficient of correlation (r)
1.	Aggregate crushing value and aggregate impact value	+0.92
2.	Aggregate crushing value and Los Angeles abrasion value	+0.73
3.	Aggregate impact value and Los Angeles abrasion value	+0.78
4.	Compressive strength (dry) and aggregate crushing value	-0.28
5.	Compressive strength (dry) and Los Angeles abrasion value	-0.25
6.	Compressive strength (dry) and compressive strength (wet)	+0.85
7.	Bulk density and compressive strength (dry)	+0.80
8.	Percentage voids and percentage water absorption	+0.6
9.	Percentage voids and compressive strength (dry)	-0.31
10.	Specific gravity and compressive strength (dry)	+0.35
11.	Compressive strength (wet) and aggregate crushing value	-0.10

TABLE 13

Mechanical properties of laterite with service behaviour

Sl. No.	Name of quarry	Mechanical properties			Service behaviour		Traffic tons	Rain-fall inches
		Impact	Per cent crushing	Abrasion	Base course	Wearing course		
16	Akia Mogra Quarry—Mhow Neemuch road M. No. 118	34.6	38.7	46.0	Very satisfactory 70 yrs.	Ruts under iron tyre	265	35
6	Sunagiri Quarry	28.2	31.4	33.4	Satisfactory 3 yrs.	—	70	150–200
1	Kesavaram Quarry, Andhra	23.7	27.9	34.1	Satisfactory 6 yrs.	Not satisfactory for heavy traffic, but satisfactory for village roads	1412	—
13	M. 164/2, Bombay Kankan-Goa Road, Ratnagiri	41.8	37.7	43.7	Satisfactory	Unsatisfactory	250	175
27	Pallapparam Quarry. M. 18/8, Palghat Shornoor road	46.0	47.0	—	Satisfactory	Unsatisfactory	1619	100–120
11	Dinod Quarry Ankleshwar-Taluka-Mansrole. 12 miles	34.0	29.0	18.0	Satisfactory 3 yrs.	—	200	45

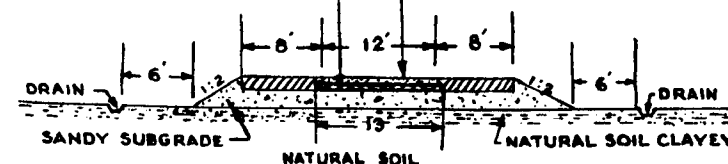
Appendix 1

Questionnaire

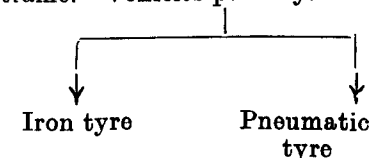
ANSWERS

1. Name of the quarry from which the laterite sample was sent.
2. Typical cross section of the road from the ground level to the riding surface clearly showing the type of construction, that is, for sub-base, base or surfacing, for which the material in question is used. (A typical rough sketch is shown below).

WATER BOUND MACADAM USING
LATERITE 3" THICK
6" THICK LATERITE SOLING



3. Type and daily volume of traffic. Vehicles per day.



4. If the material is used for wearing surface, what is the usual cycle for renewal?
5. If the material is used for soling, what is its approximate period of service so far?
6. Approximate annual rainfall in inches,

7. Based on the experience with this laterite as road building material.

- (a) As base (soling)—Is it satisfactory or unsatisfactory?
- (b) As surface course (water-bound macadam) Is it satisfactory or unsatisfactory?
- (c) Is there any bituminous surfacing provided on top in case this laterite is used as wearing course?

ACC HONOURS THE CODE

3

"Employers owe it to labour to recognise that welfare is not conceived in terms of philanthropy, but as a social obligation. Men and women engaged in production shall do so with dignity, honour and a sense of security. Fair wages should be paid for work done. Working conditions shall be as pleasant as possible."

Code of conduct—**FORUM OF
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As an employer of labour the ACC can well point with satisfaction to the salary/wages and other emoluments paid to its employees and to the many welfare activities voluntarily undertaken over the years.

To take salary/wages first, compare these figures of ten years:

	1945-46	1955-56
No. of employees	17,500	29,000
Salary/wages & amenities	Rs. 127.3 lakhs	Rs. 421.5 lakhs

Thus while the increase in the number of employees was 66%, the increase in the salary/wages, etc. for the same period was over 230%!

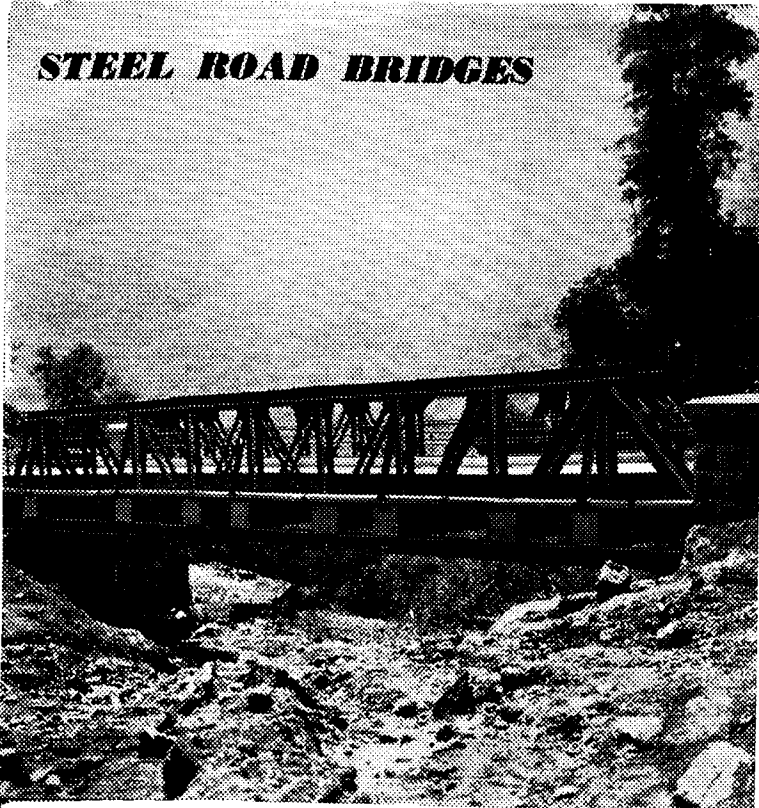
As for welfare activities, employees of the ACC's 17 factories and two collieries have for a long time been enjoying a large number of amenities for their security and social welfare, namely: provident fund and gratuity; schools, hospitals and dispensaries, creches, canteens, cinemas, clubs, playing grounds; welfare and benefit fund, and co-operative societies.

All these benefits have been provided out of a consciousness of social obligation. The ACC believes that a well-paid, well-looked-after, happy and contented employee is the best employee.

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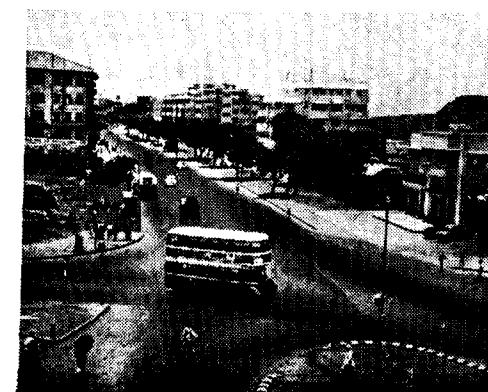
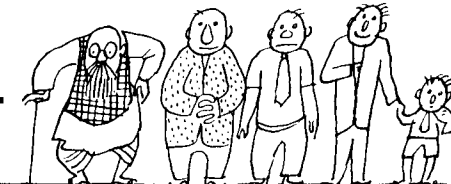
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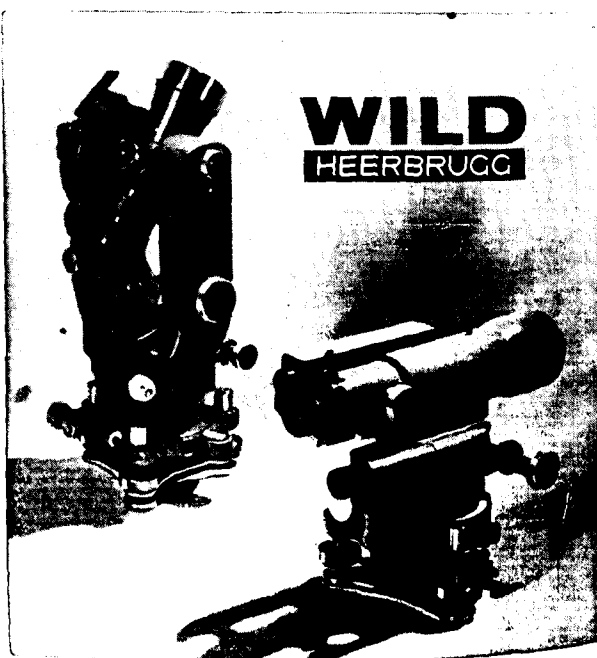
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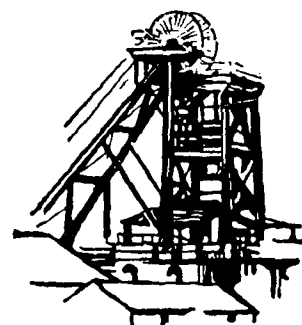
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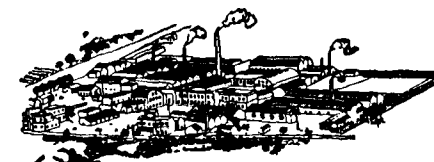


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