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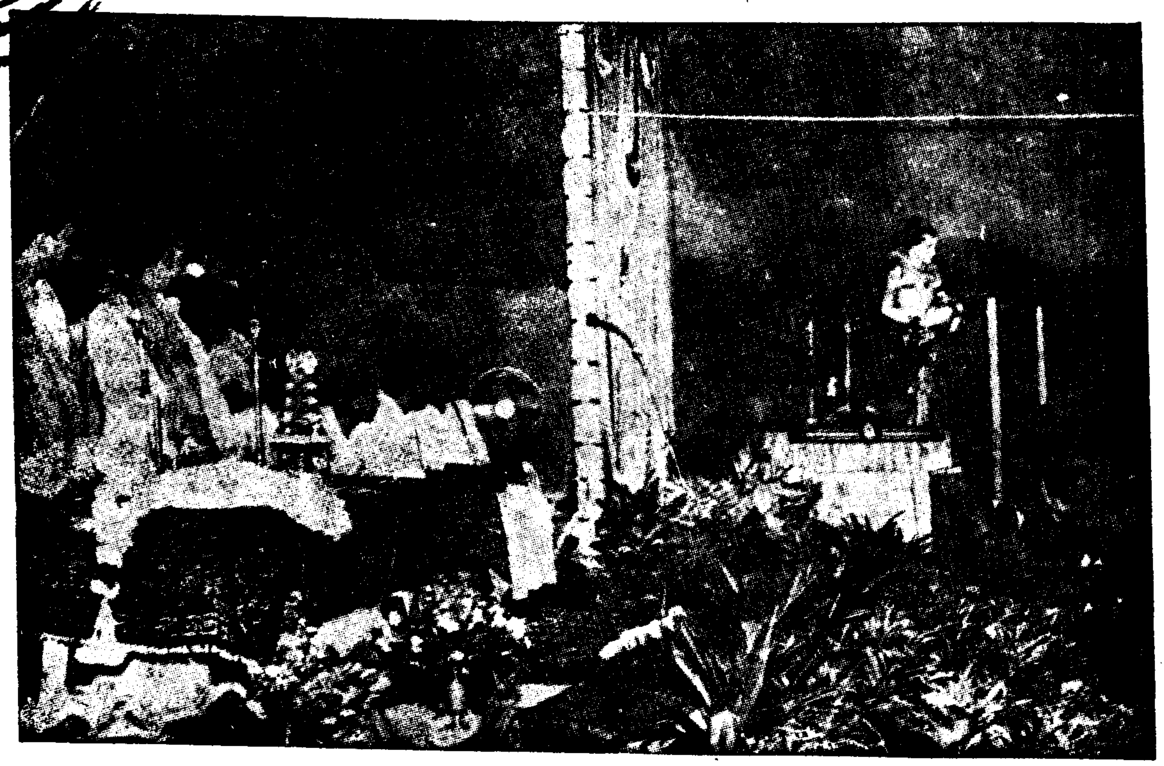
409

H.E. THE GOVERNOR OF MADRAS VISITS THE P.W.D. STALL IN MADRAS PAVILION AT INDIAN INTERNATIONAL TRADE AND INDUSTRIES FAIR

SCHEME FOR SUPPLY OF CAUVERY RIVER WATER TO MADRAS

INAUGURAL ADDRESS BY HON'BLE CHIEF MINISTER



INAUGURATION BY HON'BLE CHIEF MINISTER

THE NEW IRRIGATION ERA

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FOUNDATION TREATMENT FOR DAMS.

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1. General Conditions—

The term foundation used herein will mean all that part of area under and adjacent to the dam. The bearing capacity and the impermeability of the rocks are generally sufficient but difficulties arise due to faults, open or filled (fractures formed as a result of differential movements) crush or shear zones, joints (fractures formed without appreciable movement) bedding seams, dykes, sharp folds, zones of deep weathering and alteration and solution channels, cavities, lava tunnels and buried gorges. These various defects of rock are illustrated in figure 1 and suitable steps must be taken to rectify the defects.

1.1. *Main consideration.*—Assuming that the preliminary investigations have indicated that suitable foundation is available, the following are the main considerations for preparation of the same.

(1) Bearing capacity,

(2) Bond (with masonry),

(3) Sliding or frictional resistance,

(4) Control or reduction of uplift pressure,

(5) Prevention or control of seepage, and

(6) Prevention or control of scour from overflow or outlets.

The general nature of these considerations are explained in this section.

1·2. *Bearing capacity.*—It indicates bearing strength over the entire area besides the compressive strength of the rock itself. In general, most firm, non-decomposed rocks have sufficient bearing capacity for dam foundations but special considerations are necessary for high dams. Seamy, fissured, faulted and stratified rocks require special treatment. Treatment of these are outlined in subsequent paragraph.

1.3 *Bond.*—To obtain a tight bond between the rock and the masonry, the rock surface should be absolutely clean in addition to having a serrated and ragged surface.

1.4. *Sliding or frictional resis'ance.*—On firm, hard rock, where a tight bond can be secured, there is no danger of sliding at the contact surface. One important point to be borne in mind is that there should be as much resistance to sliding below the surface as at the surface. This makes it necessary that special consideration should be given to stratified rock and rock with open, soft or clayey horizontal seams. In extreme case it may be necessary to set the dam deep into foundation to secure a 'toe hold' in order that the weight of sufficent bed rock downstream of the dam may be available to resist the sliding forces.

1.5. *Control or reduction of uplift pressure.*—Uplift will depend on the character of the foundation rock. It is possible to control and reduce uplift pressure by grouting foundation seams and combination of grout curtain and drainage.

1.6. *Prevention or control of seepage.*—Most of the measures adopted to control uplift pressure will also work for reducing leakage except the drainage which tends to increase it.

1.7. *Prevention or control of scour.*—The hydraulic jump, principle in the design of protection features, is the basis of the more common methods of treatment. Where a deep tail water is assured, very little may be required in the way of special foundation treatment. If the tail water depth is not sufficient protection, then the need for a concrete apron is indicated. Sometimes, when conditions are not severe, a comparatively thin concrete

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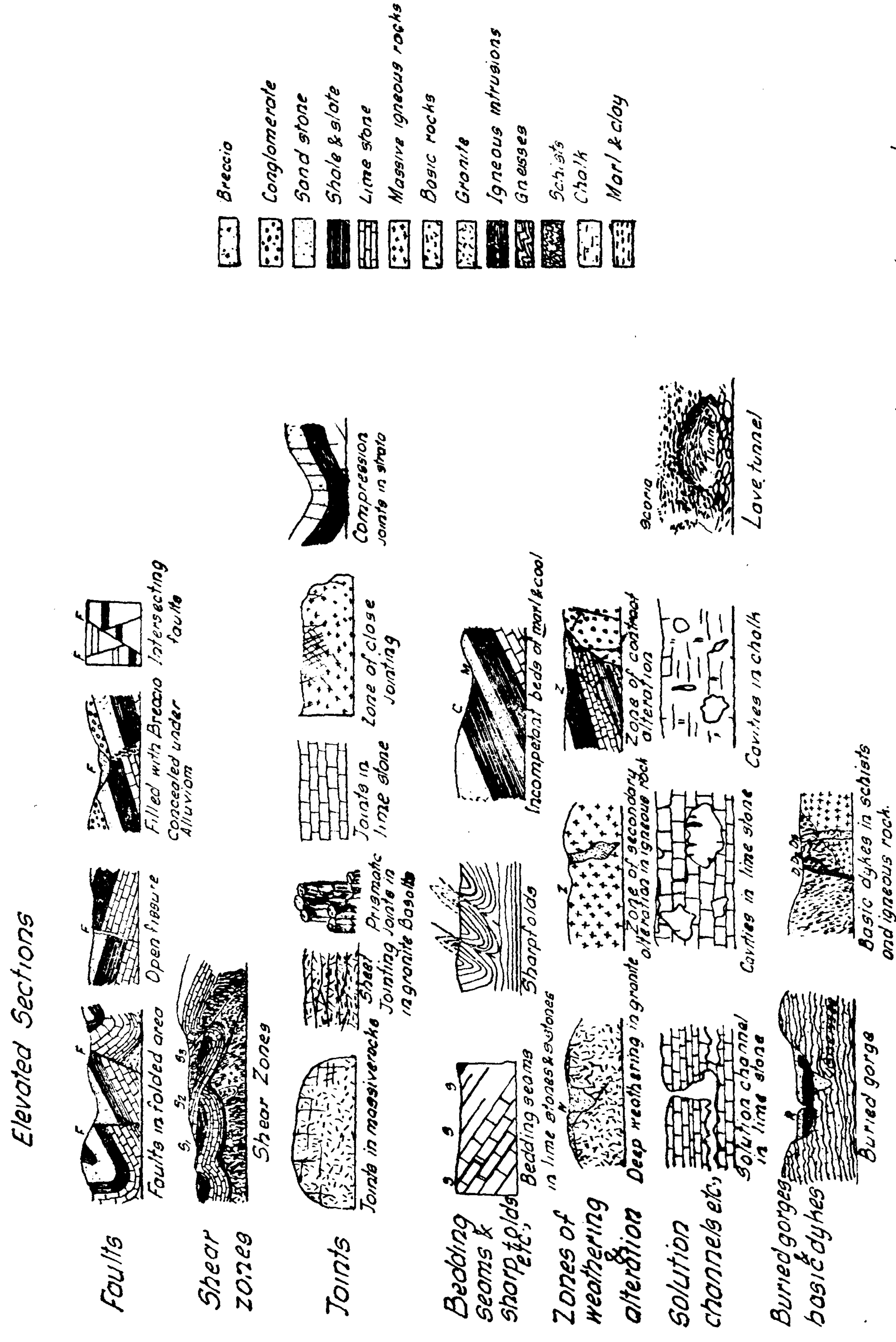


Fig 1. General defects in foundation rocks

apron will blanket the rock seams and thus serve as sufficient protection. All such aprons should be so designed as to safely resist possible uplift pressures.

2. Investigation—

Preliminary surface and sub-surface explorations are essential. The aid of a competent Engineering Geologist working in co-operation with the Engineer cannot be over-emphasized. The observations and studies should be continued even after the rock had been exposed and should include detailed surface, and sub-surface examinations up to the time of its final preparation. More detailed explorations of the surface conditions in the form of test pits, trenches, abutment drifts and core drilling should follow the preliminary exploration. To compress as a result of these explorations the Engineers in charge should have a fairly complete picture of the sub-surface with all running faults, fissures, seams, etc.

2.1 *Exploratory Drilling.*—Various types of core drills are available, viz. the diamond drill, the short drill and some new types with special steel alloy cutting edges. The diamond drill is the best.

Inclined holes, even though they are somewhat difficult to drive and cost a little more per linear foot than a vertical hole, furnish information for a greater width ; but for a higher percentages of exploratory coverage a drilling pattern which combines vertical as well as inclined holes may be found suitable.

The major facts to be observed and to be had in mind in the process of core drilling may be listed as follows :—

(*a*) Depth of over burden, (*b*) depth to ground water table, (*c*) depths at which drilling water escapes, (*d*) character and classification of rock, (*e*) condition and quality of rock (*f*) evidences of processes affecting the rock, such as crushing, weathering or other forms of decay, and solution effects, (*g*) non-recovery of core and the reasons therefor, (*h*) porosity and permeability of the rock and seepage conditions in the rock structure as a whole, (*i*) any required tests to show depths and rates of leakage into hole from ground water and (*j*) any required tests to show rate of leakage out of hole under drilling pressure or higher testing pressure.

The purpose of core drilling is to secure as nearly as possible undisturbed samples to predetermined depths of the foundation rock. Core interpretation should be done scrupu ously with the aid and under the expert supervision with an experienced man with a knowledge of engineering geology and also of the technique of drilling and core recovery. To sum up core drilling calls for expert interpretation without which the potential value of core drilling work is largely lost. Further the method and success of foundation treatment depends solely on the complete and correct interpretation.

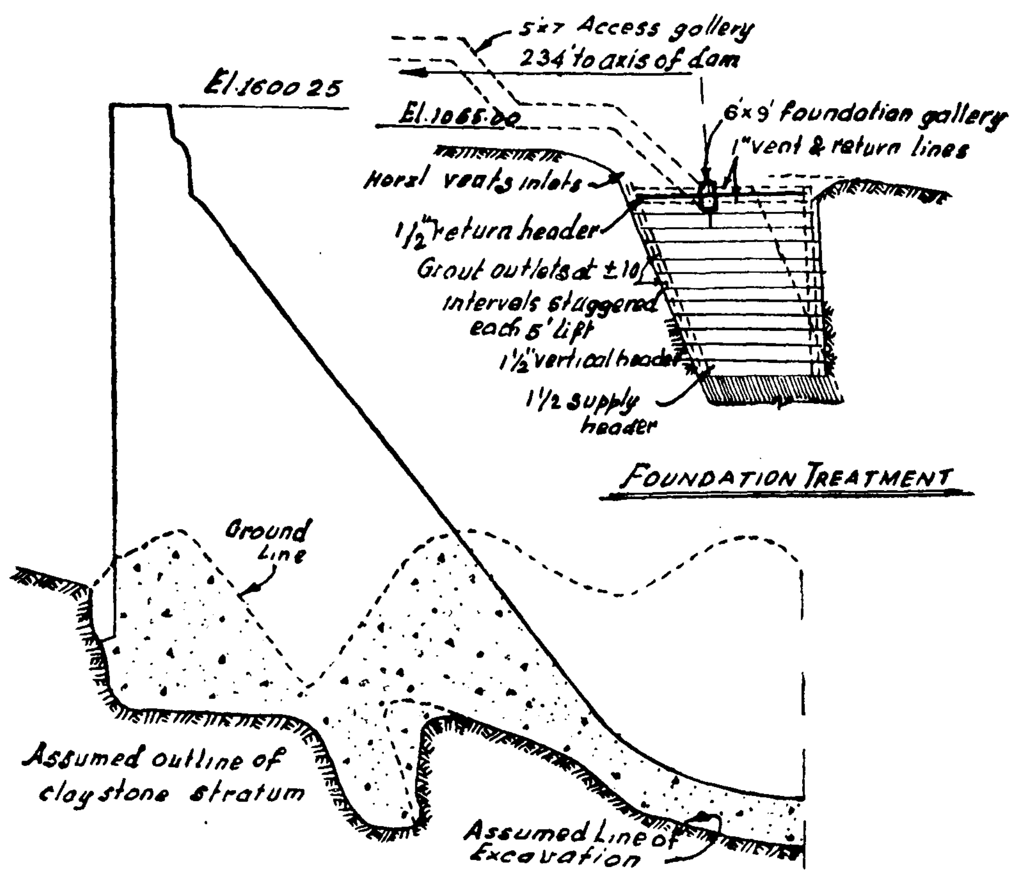
3. Excavation Work Proper— 409

Excavation work proper can be broadly divided into two categories, namely, rough excavation and final excavation.

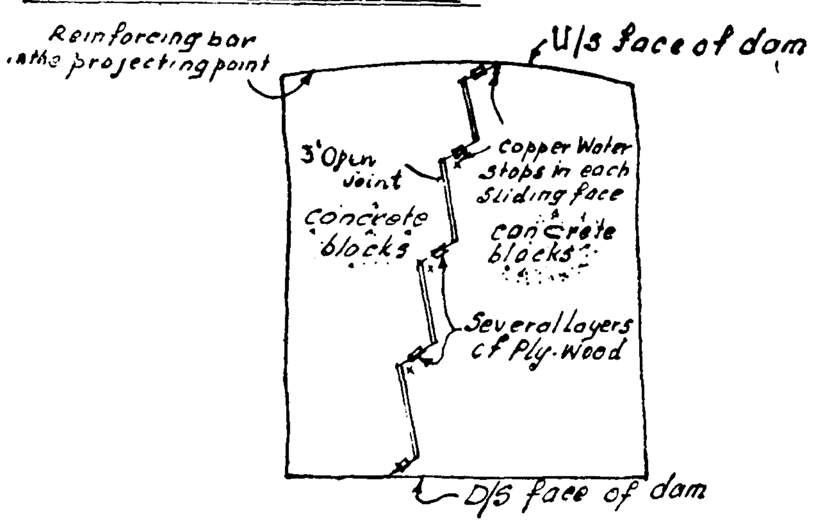
3.1 *Rough Excavation.*—The term rough exacavation refers to the removal of all over burden and of those portions of the immediately underlying rock that are soft, weathered badly, fissured or otherwise unsuitable for foundation purposes. A good practice in excavation of dam foundation is to require that the overburden be removed sufficiently to permit an early inspection of the rock. before its actual excavation is begun. The one important point is that the dam structure be founded upon solid rock entirely free from loose material, open seams or other objectionable defects. It is necessary, therefore to limit the extent of permissible blasting. An usual rule is to limit the depth of blasting hole to about two-thirds the total estimated remaining depth of excavation with progressively decreasing charges of explosives, until within a foot or two of the foundation level, when blasting is prohibited and the remaining material removed by barring, wedging, channeling, line drilling, broaching, use of paving breaker, etc.

3.2. *Final excavation.*—This is termed as 'scaling and trimming', in other words and consists of removing pieces of rock loosened by the work of rough excavation and partly

i. BAKRA (E. Punjab)

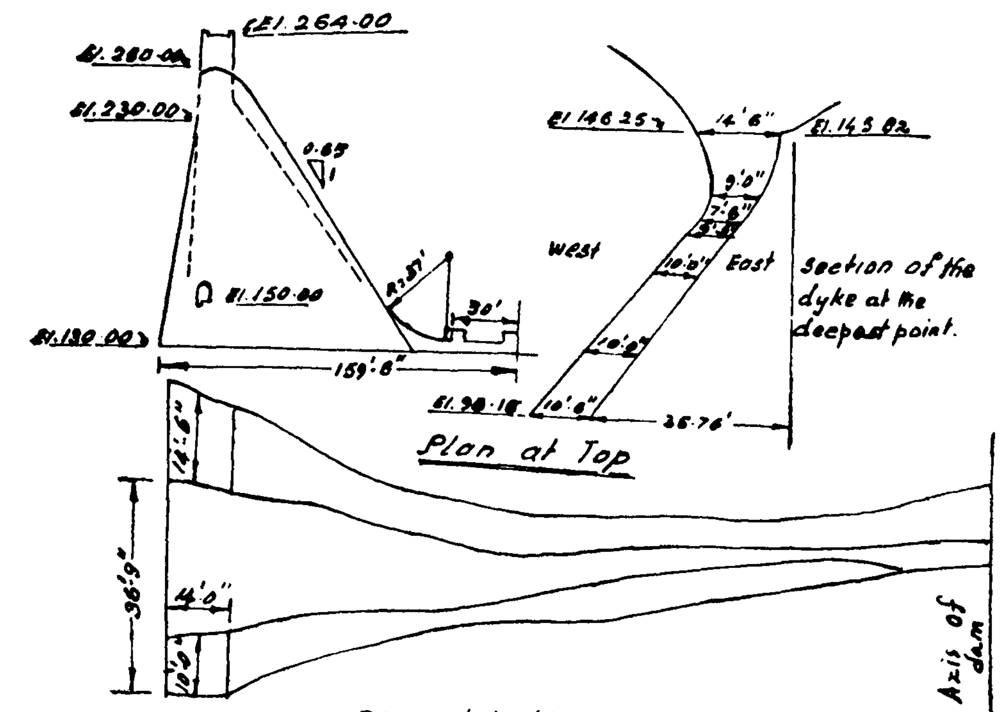


iii. MORRIS (California)



PROVISION OF OPEN-JOINT OVER FAULT

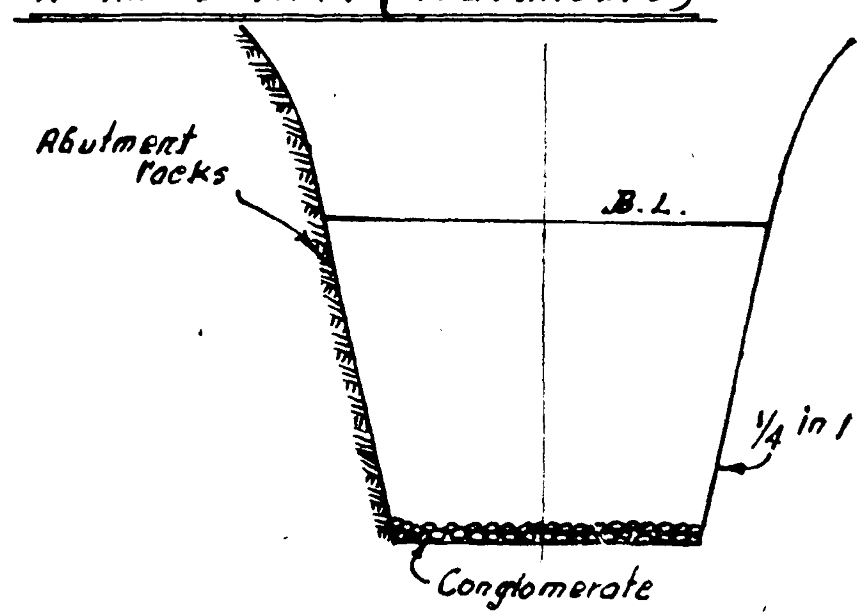
ii. PEECHI (Cochin)



Plan at bottom

Grouting

iv. MADUPATTI (Travancore)



Back Filling and Grouting

separated from the main rock mass by seams or cracks. This work should be done with great care so as not to leave on the foundation surface any rock other than that which is an integral part of the rock mass.

The prepared foundation should be free from sharp angles which might tend to induce shrinkage or shearing stresses in the concrete of the dam. Top edges of the benches or the like, should be chamferred approximately to 45°. At abrupt changes in elevations of foundation, concrete may be laid in short lifts against the rock face so as to permit initial shrinkage to take place in the lower lifts before the higher ones are placed. The possibility of having a vertical joint at these abrupt change will merit serious consideration as they lessen the possibility of shrinkage cracks and excessive shrinkage stresses.

Following the scaling and trimming the foundation surface should be thoroughly cleaned of all loose particles including the very finest of the chips, sand and dirt. For this generally jets of mixture of air and water under considerable preassure are used. As an added precaution to obtain proper bond it is sometimes specified that the finished surface shall be covered with an inch of rich mortar before the structure is built. Where it is essential to adhere strictly to prescribed line as in the case of cut offs or toe holds, line drilling or channelling and broaching (in shale or softer rocks) should be resorted to.

4. Treatment of defects—

4.1. *Seams and faults.*—As already stated, rocks are not always sound in all places free from faults, shear zones, joints, bedding seams, etc. These various defects of the rock are to be treated suitably.

At Bakra dam site, there were numerous thin shear zones varying in thickness from 6″ to 15″ laid nearly horizontal with a slight downstream dip existing in both abutments. These thin fault zones were mostly of decayed rock width clayey material infiltrated in the closely crisis crossed joining pattern. After excavating the abutment to initial excavation lines, these zones have been trenched as deep into the abutments as practicable without cutting or disturbing the rock formations on either side of the strata. These trenches were then plugged with concrete. These trenching and back filling of concrete were done in small sections of 20′ length or less. After the concrete in plug has set, drill holes of " Nx " size, at one foot intervals at specified depths and inclination were drilled through the concrete plug centrally. Then washing of seams and grouting as per the specifications are done. (Figure 2.)

At Peechi dam (Cochin) 130 feet high, there was a dyke running across the dam width filled with soft disintegrated rock. The dyke was 1′0″ wide for some length and became 20′0″ wide in the rear portions. The soft rock was removed for a depth of 56 feet and while its width decreased, it did not disappear. The rock was completely porous and the water was coming up as rapidly as it was being removed. Suggestions were made to resort to chemical injections, back filling and high pressure cement grouting.

At Madupatti (Travancore) the two abutment hills were running practically parallel and vertical, the intermediate length being filled up with big boulders with clay binding. Though the dam is not high, it was thought necessary to remove the over burden to 60′ to 80′ below the surface.

At Lower Bhavani (Madras) the rock consisted of horn blende gneisses with ultra basic intrusions giving rise to pockets of talcschists. The width of schists was at places only 1′0″ thick while at others it is 10′0″ and more. The vein was dipping up stream at an angle of 50° to 65° so that it was anticipated that it would do no harm if it was covered by atleast 10 feet of hard rock. But it was thought that talc could be avoided altogether and the alignment of the dam was shifted during execution by 70′0″ on one side and even so, a skirting channel of weathered rock was found. This had to be removed to a depth of nearly 60 feet below the bed and backfilled with concrete, though in the original designs, the anticipated depth to sound rock was taken as only 30 feet.

Tungabadra Dam is founded on epidiorite at a depth of 15 feet below the bed of the river. The rock was dipping upstream and is in slaty formation. The fault zone in this case consisted of a clay seam of varying thickness from ½″ to 1 foot (maximum thickness being in the rear). The seam continued for a length of 51 feet from the axis of the dam to the rear. A cut off trench was provided at the upstream end of the fault zone. Grout holes 30 feet to 50 feet were drilled to intercept the fault. These were used for washing the fissure and the subsequent grouting.

Washing was done by applying air and water under pressure and openings were formed initially and 4″ metal pipes were fixed in the formed openings and drilling was done through these up to 50 feet depth using calyx drills. All the formed openings were plugged by filling with concrete and futher washing was tried to create new openings along the seam. This operation was successful and after completion of washing, all the formed openings were cleaned and concreted leaving a few pockets open which would serve as ventways during actual grouting.

At Kundaly dam, there was a big cavity filled with clay inside the rock in right abutment and this was excavated and refilled with concrete.

In seismic regions, care should be bestowed on the presence of faults even though they might have been dead and inactive for considerable periods. Thus in Norris Dam a fault was encountered across the width of the dam and possible future movement due to earthquakes or other reasons provided for by an open joint. (Fig. 2).

In Conchas Dam founded on Shales, both laminated and uncemented types, care was taken to prevent rapid weathering of uncemented shales by coating the shale with water proofing preparation (Asphalt) and to prevent sliding on smooth bedding plane, a reinforced anchor wall was constructed.

In case of Norris Dam, no attempt was made to wash unsound materials from the seams as it was felt that the small size of the 2-5/16″ holes spaced so widely precluded the possibility of applying sufficient volume of water to do any real good. A complete replacement of unsound material with grout was not sought but rather a consolidation of this material by penetrating and filling the open spaces with hard grout injected under sufficient pressure to compact the loose material in the seams.

Throughout the length of Wheeler Dam, there were approximately 60 vertical joints crossing the foundation almost normal to the axis. On the surface they were ½″ to 3″ in width, widening to approximately 12″ where they intersected bedding planes were clay-filled and water bearing, and at a depth of 25 feet to 30 feet, picked out to a thin quartz-filled, water-tight joint. Near the upstream face of the dam, a 36″ short drill hole was drilled across each joint, deep enough to remove the open area. When these holes were filled with concrete, a perfect cut off plug was made.

In 1924, in the case of the great falls leakage problem, a tunnel was driven parallel to the axis of the dam along certain seams near the contact of the two Fort Payne members and same grouting was done from this tunnel. A material reduction of leakage was realised at that time. Both cement and asphalt grout were pumped into the leakage area, the former unsuccessfully, but the latter with the marked success. This leak No. 5 was almost entirely corrected with asphalt grout. Similar method was adopted in the case of Wilson Dam.

In case of Chickmuga Dam, without washing, these holes were grouted with a low cost " prescription grout " in which sand, bentonite and cement were used. The pressure used were generally 0 5 psi for each foot of overburden or if in deep rock, 1 psi was added for each foot of depth in the rock.

The pervious foundations, met in Mula Dam site in Maharashtra State, are proposed to be made water-tight by the treatment of the deeper layers by grouting while for the upper part of the foundation, the natural soil will be excavated and removed and back filled with clayey soil.

The process of grouting consists of two stages: (1) Using clay cement grouting; (2) Using bentonite silicate grouts, for grouting; the pockets of medium sand which do not accept clay cement grout.

To sum up, narrow seams and faults can be washed out and grouted. Wide seams can be cleaned and plugged with concrete. If such a seam is found in a nearly horizontal plane below the level of final excavation it is usually better to reach it by a vertical shaft or a large size drill hole and clean out and plug with concrete, than to excavate the firm rock above. Cavernous rock and solution channels may also be treated similarly.

In the case of low dams some small areas of weak rock are left in place on the assumption that the dam will span over them. Sometimes vertical faults are filled with concrete just to that depth as to have an arch spanning the gap. In all these cases there is bound to be concentration of stresses in the masonry adjacent to the opening and the magnitude will have to be assumed at least roughly before deciding on such a treatment.

4·2. *Leakage.*—Some leakage through rock is to be expected as there is no completely impervious rock with no clevages or breaks whatsoever. Two methods are available to confine leakage to a reasonable limit viz., cut off trench and grout curtain cut offs. The former is better of the two but the latter is cheaper. These cut off tend to reduce uplift on the dam also.

4·3. *Grouting and drainage.*—The purpose is to reduce leakage and uplift and to consolidate the foundation. The latter is called consolidation grouting to differentiate from the former.

Grout is composed of a mixture of neat cement and water with admixture like rock flour, bentonite, clay etc., under special circumstances. A thorough study of the geological feature should precede any grouting operation.

4·4. *Pressure and pressure control.*—Pressures from 50 to 150 psi is named as low pressure grouting and applied up to 40 feet depth and pressure from 200 to 600 psi is named as high pressure grouting. Pressure used for grouting should be carefully limited to that which will not lift or otherwise move any part of the foundation or structure thereon. The pressure must be great enough consistant with speedy operation and wide coverage. Creager Justin and Hinds recommended the following equations as rough guide for determining the maximum permissible pressure.

Let "p" be the maximum allowable pressure in lbs. per sq. inch at 'h' feet below surface then—

for massive rock—

$$p=h+1{\cdot}33h\left(\frac{h}{100}+\frac{3\sqrt{h}}{20}\right)$$

for sound stratified rock—

$$p=h+1{\cdot}33h\left(\frac{h}{900}+\frac{\sqrt{h}}{20}\right)$$

for sound stratified rock which has been grouted above the given elevation—

$$p=h+1{\cdot}33h\left(\frac{h}{400}+\frac{3\sqrt{h}}{40}\right)$$

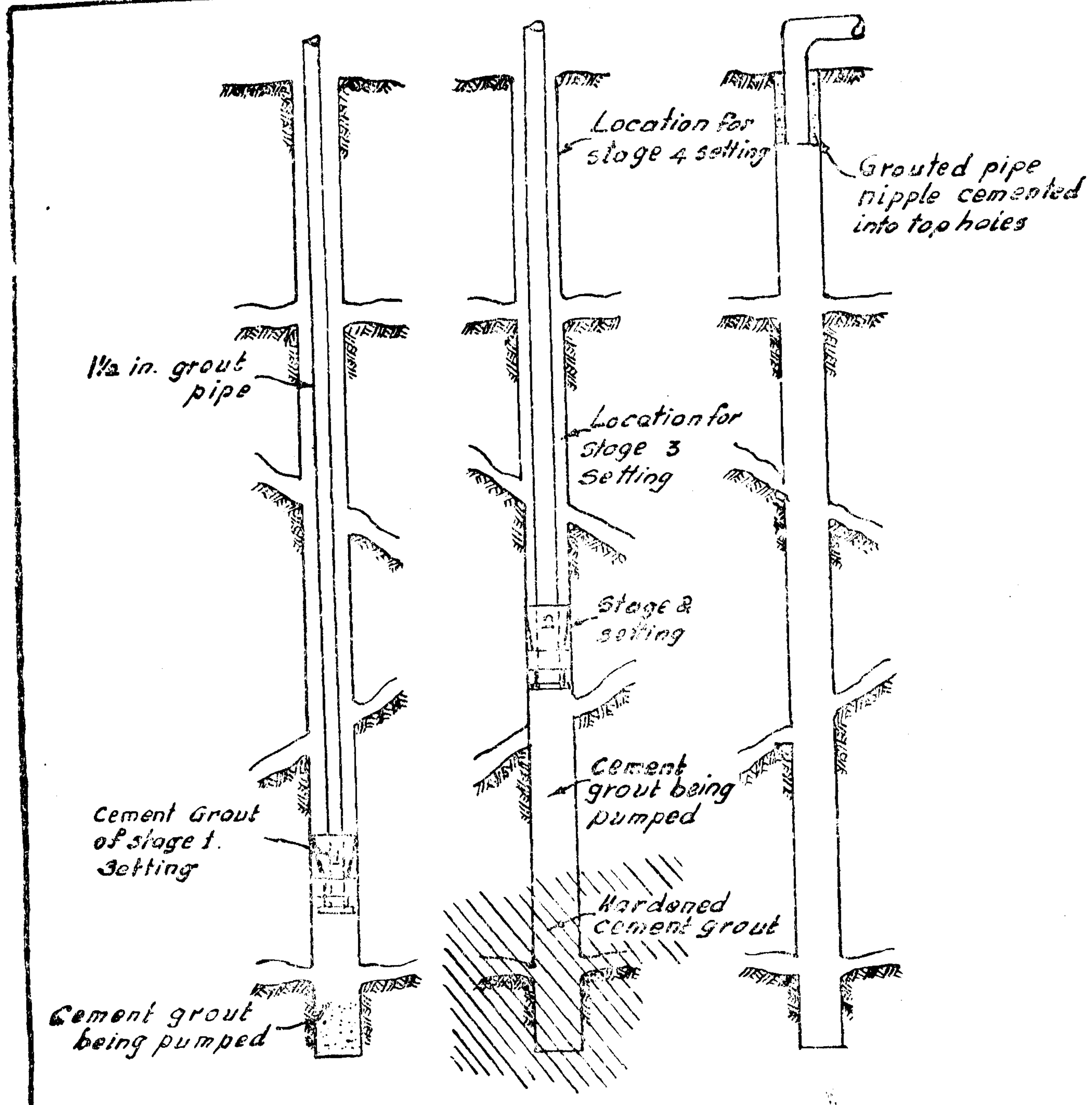
There is another general rule which states that the pressure of grouting in lbs per sq. inch at any elevation should not exceed the depth in feet.

To determine the depth of holes the following rules are sometimes employed in practice.

(1) The depth of hole should not be less than one-fourth the hydrostatic head at the maximum water level behind the dam.

(2) The depth of hole should be one-third the maximum height of the dam above the hole, plus a constant varying from 25 ft. to 75 ft.

4·5. *Washing out.*—Each grout hole should be cleaned out thoroughly with compressed air and water. Water should then be pumped into holes at moderate pressure to clean out wide seams and to locate leaks to be plugged and to test the rate of flow. Narrow seams filled with gauge or clay need not be cleaned.



GROUTING BY STAGES & BY USING A SINGLE SETTING

FIG 3.

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4·6. *Grouting programme.*—Thin grout travels farther than thick grout. At the start of grouting operation thin mixture of one part of cement to five or six parts of water is used which is gradually thickened ultimately to 1 to 1 or 1 part to 0·6 part of water. Curtain grouting is accomplished by grouting one or several lines of holes at the upstream face of dam. Generally more than one line of holes will not be required except in cases where exceptionally high pressures are required. In such cases, additional lines of holes are shallower and are utilised to consolidate the surface over a wider area. Sometimes 'split spacing' is adopted, where primary series holes are drilled 15 feet to 25 feet apart. The second and third series of equal number of holes are drilled intermediately to reduce the final spacing to quarter of the first. It is preferable to grout each hole before the adjacent holes are drilled.

4·7. *Stage grouting.*—Where the foundation contains weak horizontal bedding planes, that are apt to be lifted up by grout pressure and where high pressures are required for the lowest depths 'Stage grouting' is resorted to. One method of doing this is to drill a shallow hole, grout at low pressure and clean out before the grout has set very hard. After the grout in the rock has set the hole is drilled deeper grouted under higher pressure and the process repeated till the required depth and pressure is reached. Another method is to drill holes to the final depth and extend the pipe to the top of the lowest zone, use an expanding seal at the end of the grout pipe to prevent the grout from rising and grout at high pressure. The pipe is then raised so that the seal is at the top of the next zone to be grouted at somewhat lower pressure.

Usually the tightness of the foundation increases with depth and it is not necessary to carry all holes to the deepest depths. In these cases one or more lines of shallow holes, from 10 to 20 feet deep are drilled and grouted first, in order to consolidate the surface and to prevent leakage of high pressure grout to be used at lower elevations. Then another single line of hole is drilled at wider spacing and grouted at higher pressure and so on.

4·8. *Special grouting.*—Special grouting is often required on steep abutments, where major surface seams crevices or fractures in the rock are in evidence where percolation path may be relatively short. Such special grouting with properly placed concrete cut off walls will be effective means to prevent undue seepage through and around abutments.

4·9. *Drainage.*—The purpose of drainage is to relieve uplift pressure, thereby increasing the stability of the dam and to pick up and safely conduct to the river channel below any seepage water that may find its way through or beneath the dam. It is accomplished, usually, by means of a series of drainage holes 3″ to 6″ in diameter, about 10 feet on centres and drilled into the foundation rock to depths of from 25′ to 50′ or more. The holes are located on a line parallel to and a short distance downstream from the grout curtain. The seepage water wells up through the drainage holes, usually into a drainage gallery from which it is led away. While constructing the base of the dam metal pipes may be installed or holes formed otherwise, to facilitate subsequent drilling of the drainage holes. These drainage holes should not be drilled until all grout holes have been drilled and grouted.

4·10. *Consolidation grouting.*—Badly faulted and cracked foundations of otherwise good rock can be successfully consolidated by grouting. The holes are seldom more than 20 or 30 feet deep. They are so spaced along the length and width of foundation so as to form regular hexagons with central holes. It is very essential that all the seams be thoroughly washed out before consolidation grouting begins.

5. Treatment of foundation for earth dam —

5·1. *General.*—It is possible to construct an earth dam on almost any foundation available. Foundations of earth dams are often more or less recent alluvial deposits which have not been consolidated under any material load. Coarse sands and gravel in the foundation give no trouble with regard to stability. For very fine and uniform sands great caution is necessary. It is possible to consolidate such a foundation by pre-saturation or blasting.

A plastic clay foundation is the one which will require the greatest amount of study and investigation. Frequently extremely flat slopes must be used for the earth dam built on such a foundation in order to keep the stresses sufficiently less than the strength of the material to provide a suitable factor of safety.

The treatment proposed for different depths of pervious foundation are all given in a tabular statement together with the connected drawings which are self explanatory.

5·2. *Treatment of foundation*—(i) *Clearing and stripping.*—Trees and stumps should be removed from the site. Requirements of stripping vary with the conditions at site. As a general rule stripping requires only removing the sod (and not necessarily all the roots and obtaining a bond between the embankment material and foundation material).

5·3. *Removal of vegetable matter.*—Some specifications require that all soil containing more than 6 per cent of vegetable matter shall be removed from the site. The percentage of allowable vegetable matter varies from three to eight. In many cases the presence of a small percentage of vegetable matter may help to make the material more impermeable.

5·4. *Bonding Dam to Foundation.*—It is a matter of prime importance to make sure that there is no definite dividing plane between the foundation material and the material composing the embankment. The original surface should be ploughed up before depositing any material on it. In the case of rolled fill embankment the surface should be liberally moistened before the first layer is placed.

5·5. *Conclusion.*—A rather complete but concise treatment has been attempted in this article on the basic consideration involved in the examination and preparation of foundation for masonry dams. It is hoped it will serve as a useful guide for personnel charged with the particular duty.

EARTH DAM—TREATMENT OF PERVIOUS FOUNDATIONS.

Case number.	*Figure number.*	*Thickness of overlying impervious layer.*	*Depth of pervious foundations.*	*Stratified or homogeneous.*	*Device for control of seepage.*	*Additional requirements (other than stripping).*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	107 (A)	None ..	Shallow ..	Either ..	Cut-off trench [sec. 126 (*b*)].	Toe drain [section 126 (i)] downstream filter may be required [sections 126 (*b*) and 127 (*b*).] Grouting may be required (section 124).
1	107 (B)	None ..	Intermediate ..	Either ..	Sheet piling or cement bound curtain cut-off [sections 126 (*d*) and (*e*).]	Toe drain [section 126 (i)] large core required [section 134 (*e*)] key trench [section 123]. Downstream filter my be required [section 126 (*h*) and 127 (*c*)].
1	108	None ..	Intermediate or deep.	Stratified ..	Partial cut-off trench [section 126 (*c*)].	Same as above, except no key trench required.

EARTH DAM—TREATMENT OF PERVIOUS FOUNDATIONS.

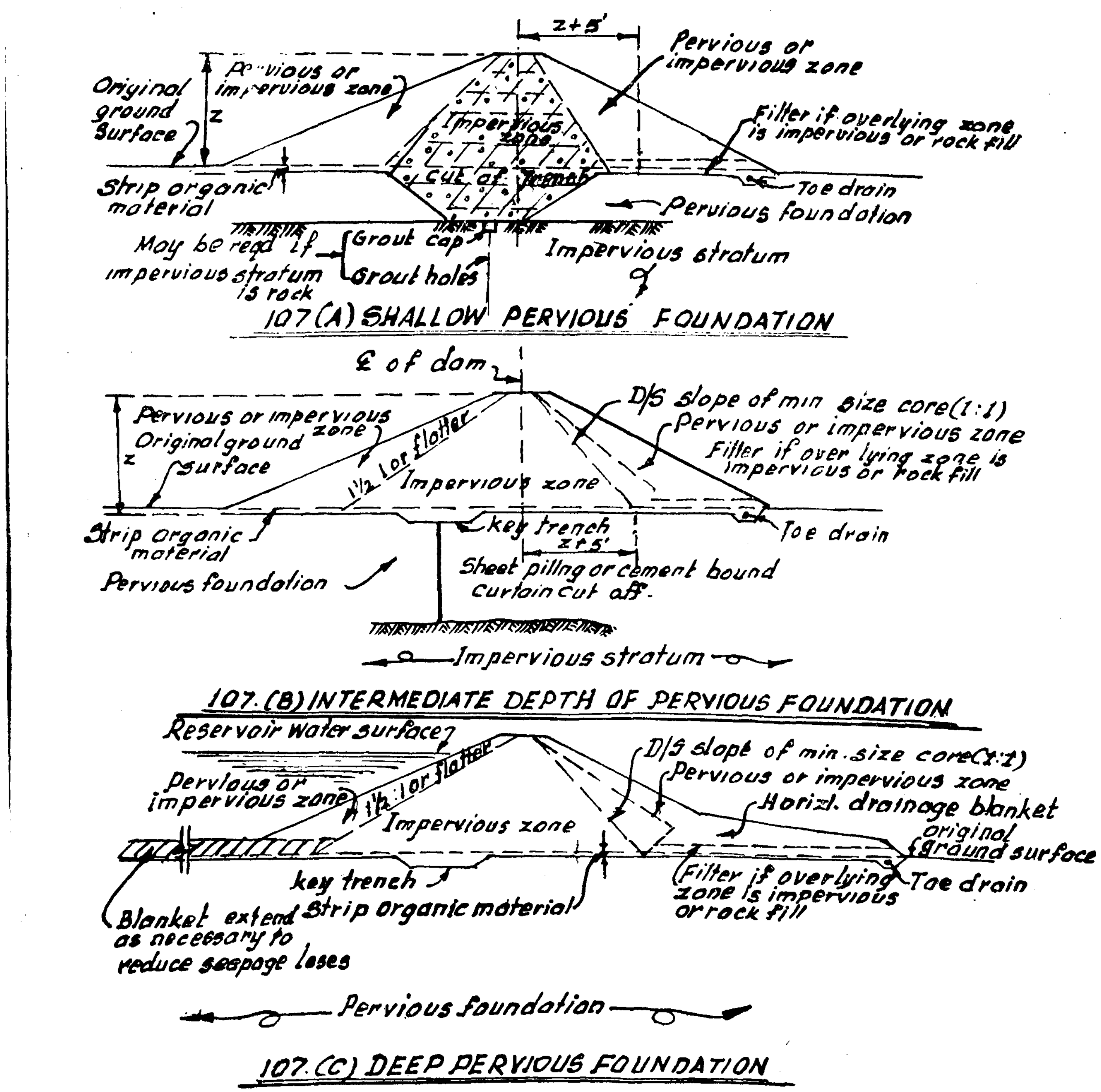

FIG 4.

Inkd. C. S. K.

Case number.	*Figure number.*	*Thickness of overlying impervious layer.*	*Depth of pervious foundations.*	*Stratified or homogeneous.*	*Device for control of seepage.*	*Additional requirements (other than stripping).*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	107 (C)	None ..	Deep ..	Either ..	Horizntaol drainage blanket [section 126 (*h*)].	Large core required [section 134 (*e*)]. Upstream blanket section 134 (*e*)]. Upstream blanket [section 126 (*g*)] may be required to minimize seepage loss. Key trench (section 123), toe drain [section 126 (i)].
2	107 (A), (B),(C), 108.	Less than 3 feet.	Various depths	....	Treat as corresponding case 1 foundation.	....
2	107 (A), (B), 108.	More than 3 feet, less than reservoir head.	Shallow or intermediate.	....	Treat as corresponding case 1 foundation.	....
2	109(A) or (B).	More than 3 feet, less than reservoir head.	Deep ..	Homogeneous.	Drainage trench [sections 126 (i) and 127 (*e*)] or pressure relief wells [section 126 (*j*)].	Key trench (section 123).
2	109(B)	More than 3 feet, less than reservoir head.	Deep ..	Stratified ..	Pressure relief wells (section 126 (*j*)].	Key trench (section 123), Toe drain [section 126 (i)].
2	..	More than reservoir head.	..	..	No treatment required as pervious foundation.	

Case 1—The pervious foundation is exposed.

Case 2—The pervious foundation is overlain by an impervious layer which may vary in thickness from a few feet to a considerable depth.

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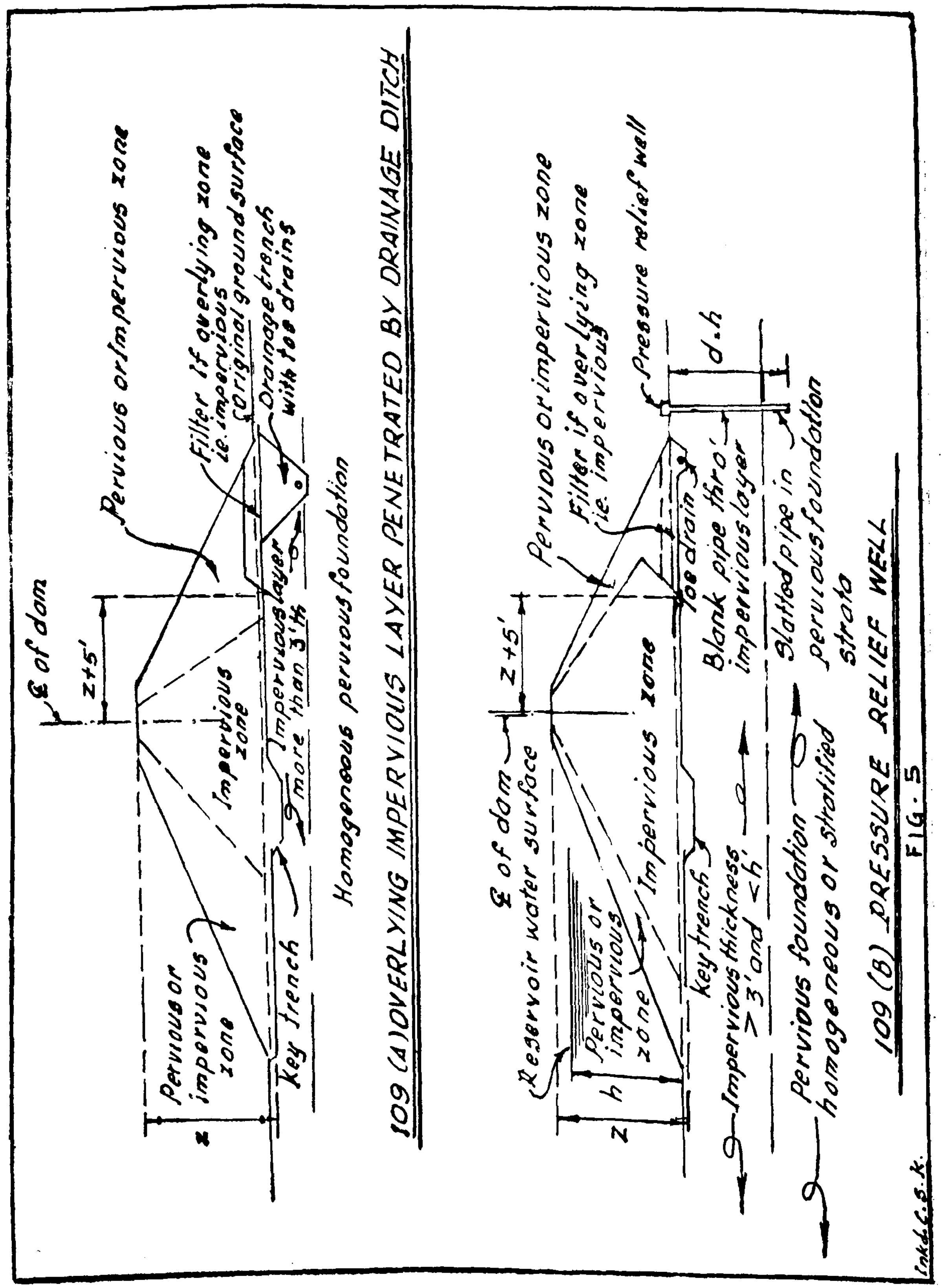

109 (A) OVERLYING IMPERVIOUS LAYER PENETRATED BY DRAINAGE DITCH

109 (B) PRESSURE RELIEF WELL

FIG. 5

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RELATION BETWEEN MANNING 'n' AND EQUIVALENT ROUGHNESS 'k'

BY

* THIRU P. A. ASWATHANARAYANA, B.E., M.E., M.I.A.H.R.

Notation—

A = Co-efficient.
C = Chezy's co-efficient.
C_1 = Proportionality factor.
C^1 = Co-efficient.
D = Diameter of pipe in feet.
F = Constant of proportionality.
f = Friction factor.
g = Acceleration due to gravity ft./sec².
K = Equivalent sand roughness.
L = Linear dimension.
n = Manning coefficient of roughness.
R = Hydraulic radius in feet.
r_0 = Radius in feet.
S = Slope of energy gradient.
V = Mean velocity of fluid in feet per second.
V_f = Friction velocity.
y = Depth in feet.
α = Exponent of hydraulic radius R
β = Exponent of slope S.
γ = $\alpha - \frac{1}{2}$

Introduction :

Long before the derivation of the Karman-Prandtl equations of velocity distribution and resistance, it was necessary to evaluate the probable loss of head from empirical relationships which took into consideration only the effect of boundary roughness. A complete list of various formulae is given elsewhere.[1] Since some of these are still commonly used in hydraulic design—particularly in the case of open channels—and since the simplicity of one, the Manning formula—continues to warrant its use in first approximations. Then Manning formula is generally written as

$$V = \frac{1{\cdot}486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{.. (1)}$$

in which n is a characteristic of the boundary material.

The Manning n was developed empirically as a co-efficient which remained a constant for a given boundary condition, regardless of a slope of channel, size of channel or depth of flow. As a matter of fact, the Reynolds number, the shape of the channel, and the relative roughness have an influence on the magnitude of Manning n. Furthermore, for a given alluvial bed of an open channel, the size, the pattern, and the spacing of the dunes vary with slope, discharge, channel shape, and character of bed material, so that n also varies. Despite the shortcomings of Manning roughness co-efficient, however, it has an amazing utility and is used extensively in Europe, India, Egypt and U.S. It has also been concluded,[2] on the basis of various discussions and comments on the subject, that Manning 'n' serves the purpose very well.

The results of systematic tests by Nikuradse on turbulent flow in smooth and rough pipes demonstrate perfectly the relationship between f the friction factor, Reynolds number and relative roughness. In these tests geometric similarity of the roughness pattern was obtained by fixing a coating of uniform sand grains to the pipe wall, thus giving an easily measurable index for the relative roughness, k (size of roughness), k being the diameter of sand grains. The essential feature of sand roughness is that the surface is covered with sand as densely as possible and that the roughness density is at its maximum. For many practical roughnesses, the roughness density is considerably smaller. It is also necessary to have standard of roughness which is fixed, reproducible and representative (at least approximately) of natural roughness. The work of Nikuradse has resulted in a standard which is fixed and is representative of many natural roughnesses, but it has not been reproducible and does not represent

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adequately certain natural roughness. Despite the inadequate features of the Nikuradse roughness, k, however, it is only generally accepted roughness standard, and until a better one is developed, it will continue to fill, at least in part, an important need.

As the use of one geometric variable, k, of closely packed sand grains, results in a convenient mathematical expression of the resistance relationship, it is usual to try to relate the Manning n with equivalent sand roughness or Nikuradse k. Also the equations relating n and k may be used effectively for interpreting n in terms of tangible quantities. For example, in the construction of models of open channels and rivers, where the relative roughness of model and prototype are to be equal, the equations are useful in estimating the value of n required for model.

Relation between (n) *and* (k).—(a) Strickler[3] arrived at the relation as follows :

Comparing the Chezy formula

$$V = C\sqrt{RS} \qquad \text{.. (2)}$$

with the Manning formula, the following relationship results

$$C = \frac{1{\cdot}486}{n} R^{1/6} \qquad \text{.. (3)}$$

Keulegan's equation[4] for rough channel can be represented in the form

$$C = 32{\cdot}6 \log \frac{12{\cdot}22\,R}{k} \qquad \text{.. (4)}$$

Eliminating C from the above two equations, Manning n may be expressed as

$$n = \phi\left(\frac{R}{k}\right) k^{1/6} \qquad \text{.. (5)}$$

where

$$\phi\left(\frac{R}{k}\right) = \frac{\left(\frac{R}{k}\right)^{1/6}}{21{\cdot}9 \log\left(12{\cdot}2\frac{R}{k}\right)} \qquad \text{.. (6)}$$

The plot of this equation indicated that for a wide range of $\frac{R}{k}$, the variation in $\phi\left(\frac{R}{k}\right)$ is small. As an approximation, $\phi\left(\frac{R}{k}\right)$ was assumed constant and equal to an average value on the basis of actual observations made in Switzerland, Strickler arrived at a formula which when compared with equation gave an average value of $\phi\left(\frac{R}{k}\right) = 0{\cdot}0342$ and thus giving

$$n = 0{\cdot}0342\ k^{1/6} \qquad \text{.. (7)}$$

(b) On a similar approach, made on composite roughness for steady non-uniform flow in open rectangular channel, Lakshmana Rao and Aswatha Narayana[5] gave the relation as

$$n = 0{\cdot}0351\ k^{1/6} \qquad \text{.. (8)}$$

(c) Advani[6] derived the relation in the following way. Relating Manning formula and friction velocity V_f

$$\frac{V}{V_f} = \frac{1{\cdot}486\ R^{2/3}\ S^{\frac{1}{2}}}{n} \times \frac{1}{\sqrt{g\ RS}} \qquad \text{.. (9)}$$

$$= \frac{1{\cdot}486}{\sqrt{g}} \left(\frac{R}{k}\right)^{1/6} \frac{k^{1/6}}{n} \qquad \text{.. (10)}$$

Also

$$\frac{V}{V_f} = \frac{C\sqrt{RS}}{\sqrt{gRS}} = \frac{C}{\sqrt{g}} \qquad \text{.. (11)}$$

Comparing Darcy and Weisbach formula which is given as

$$S = \frac{f}{4R}\frac{V^2}{2g} \qquad \text{.. (12)}$$

and Chezy formula results

$$C = \sqrt{\frac{8g}{f}} \qquad \text{.. (13)}$$

Hence

$$\frac{V}{V_f} = \sqrt{\frac{8g}{fg}} = \sqrt{\frac{8}{f}} \qquad \text{.. (14)}$$

As $\frac{V}{V_f} = \sqrt{\frac{8}{f}}$ and f, the friction coefficient, is dimensionless and depends on $\frac{R}{k}$ for rough turbulent flow, it is reasoned that $\frac{V}{V_f}$ is also dimensionless and depends on $\frac{R}{k}$.

Therefore, $n \propto k^{1/6}$ .. (15)

Also $\frac{V}{V_f} = \sqrt{\frac{8}{f}} \propto \left(\frac{R}{k}\right)^{1/6}$ .. (16)

Therefore $\frac{1}{f} \propto \left(\frac{R}{k}\right)^{1/3}$

or $\frac{1}{f} = C_1 \left(\frac{R}{k}\right)^{1/3}$ .. (17)

Comparing this equation with Prandtl's equation for rough turbulent flow.

$$\frac{1}{\sqrt{f}} = 1.74 + 2 \log_{10} \frac{r_o}{k} \quad .. \quad (18)$$

C_1 was found equal to 8·625 for values of $\frac{R}{k} \geq 4.0$.

Thus $\frac{1}{f} = 8.625 \left(\frac{R}{k}\right)^{1/3}$ .. (19)

or $\frac{V}{V_f} = \sqrt{\frac{8}{f}} = 8.307 \left(\frac{R}{k}\right)^{1/6}$ (20)

Also $\frac{V}{V_f} = \frac{1.486}{\sqrt{g}} \left(\frac{R}{k}\right)^{1/6} \frac{k^{1/6}}{n}$ (21)

giving the result $n = 0.0315\, k^{1/6}$ (22)

(*d*) Govinda Rao and Rajaratnam approached the problem in another way. Relating Manning formula and friction velocity

$$\frac{V}{V_f} = \frac{1.486}{n} R^{2/3} S^{1/2} \times \frac{1}{\sqrt{gRS}} \quad (9)$$

$$= A \left(\frac{R}{k}\right)^{1/6} \quad .. \quad (23)$$

where $A = \frac{1.486}{n} \frac{1}{\sqrt{g}} k^{1/6}$ .. (24)

For infinitely wide channel R=y

$$\frac{V}{V_f} = A \left(\frac{y}{k}\right)^{1/6} \quad .. \quad (25)$$

Keulegan's formula[4] for an infinitely wide rough channel is

$$\frac{V}{V_f} = 6.0 + 5.75 \log\left(\frac{y}{k}\right) \quad .. \quad (26)$$

Relating the above two equations

$$6.0 + 5.75 \log\left(\frac{y}{k}\right) = A\left(\frac{y}{k}\right)^{1/6} \quad (27)$$

For values of $\frac{y}{k}$ from 15 to 500, an average value of 8·0 for A was found out. From this, it was shown

$$n = 0.024\, k^{1/6} \quad .. \quad (28)$$

(*e*) Ackers[8] arrived at the relation in the following way. Writing the resistance equation in the general form

$$V = C^1 R^{\alpha} S^{\beta} \quad .. \quad (29)$$

where C^1 is a co-efficient and α and β are exponents of R and S.

In the zone of complete development of rough turbulence, the friction factor ceases to be influenced by viscosity so that f becomes a function of relative roughness only. This may be written as

$$\frac{8g}{C^{1^2}} R^{1-2\alpha} S^{1-2\beta} = \phi\left(\frac{R}{k}\right) \quad .. \quad (30)$$

For this to be single valued, the slope must be eliminated from the left side because of its non-occurrence on the right side, i.e., $\beta = \frac{1}{2}$. For the term on the left to depend on the ratio $\frac{R}{k}$, $\frac{C^{1^2}}{8g}$ must be related to k to make the exponents proportionate on both sides of the equation, thus

$$\frac{C^{1^2}}{8g} \propto k^{1-2\alpha} \quad .. \quad (31)$$

Introducing a dimensionless proportionality constant, F

$$C^1 = F g^{1/2} k^{1/2-\alpha} \quad .. \quad (32)$$

Inserting $\beta = \frac{1}{2}$, the generalised empirical equation then becomes

$$C = \frac{V}{\sqrt{RS}} = \frac{F g^{1/2} k^{1/2-\alpha} R^{\alpha} S^{1/2}}{R^{1/2} S^{1/2}}$$

$$= F g^{1/2} \left(\frac{R}{k}\right)^{\gamma} \quad .. \quad (33)$$

where $\gamma = \alpha - \frac{1}{2}$ .. (34)

For $\gamma = 1/6$ the equation becomes comparable to that of Manning Strickler with co-efficient F equal to 8·41. Therefore

$$C = 8.41\, g^{1/2} \left(\frac{R}{k}\right)^{1/6} \quad .. \quad (35)$$

Substituting this in Chezy formula and then comparing with Manning formula results

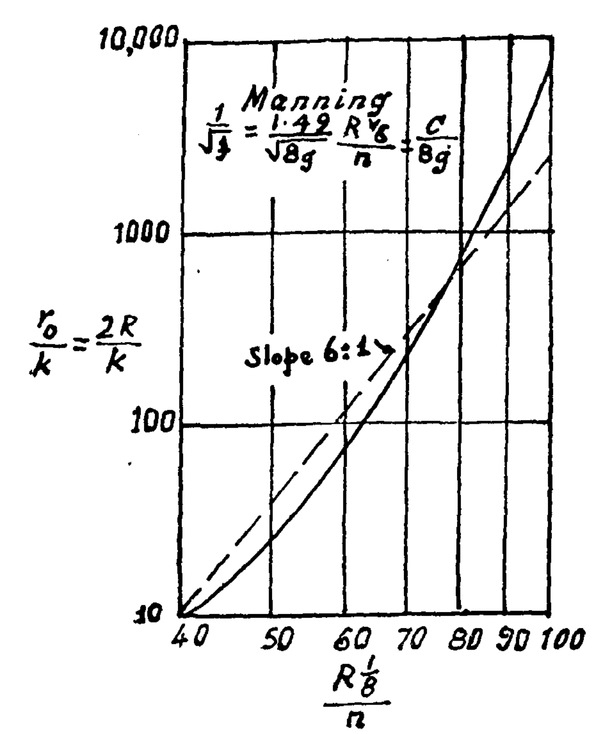
$$n = 0.03116\, k^{1/6} \quad .. \quad (36)$$

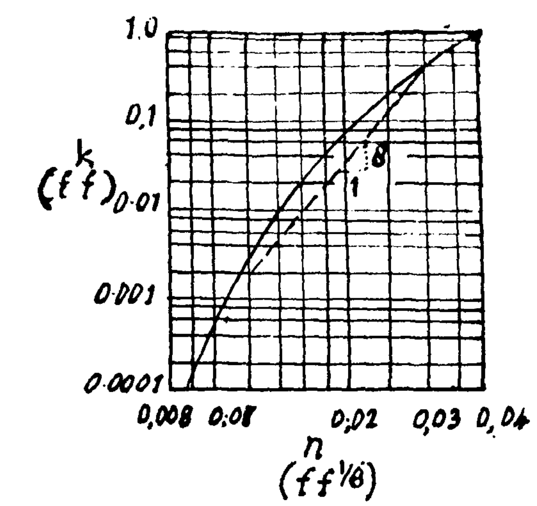
(*f*) Rouse[9] arrived at the relation in the following way :

From Eq. 3 modified as $C = \frac{1.49}{n} R^{1/6}$ and Eq. 13, we get

$$\frac{1}{\sqrt{f}} = \frac{1.49}{\sqrt{8g}} \frac{R^{1/6}}{n} \quad .. \quad (37)$$

As n varies only with the boundary roughness, the factor 1·49 must have the dimension of $\sqrt{g}$. Therefore, since the hydraulic radius R is a linear factor, it follows that n must have the dimension $L^{1/6}$, $\frac{R^{1/6}}{n}$ then representing a ratio of two lengths similar to the relative roughness. Since both roughness ratios are now related to f the Manning parameter $\frac{R^{1/6}}{n}$ is expressed in terms of the parameter $\frac{r_o}{k} = \frac{2R}{k}$ as in Fig. 1. Were the empirical formula of Manning compatible with the more nearly correct relationship of Eq. 18, the curve in Fig. 1 would be a straight line having the slope 6 : 1 — the actual deviation of the curve from this line being an error involved in Eq. 1. Hence Manning formula is dependable for intermediate values of relative roughness.





(*g*) Rouse and Howe[10] proceeded in the following way. If both Eq. 3 and Eq. 18 correctly indicated the effect of roughness upon the boundary resistance, a plot of log k against log n would have a 6—1 slope. This is approximately true, as seen from Fig. 2 which has been plotted with the help of Eq. 3, Eq. 18 and Moody's diagram. Manning formula deviates appreciably from the Karman—Prandtl equation as the roughness magnitude becomes either very large or very small.

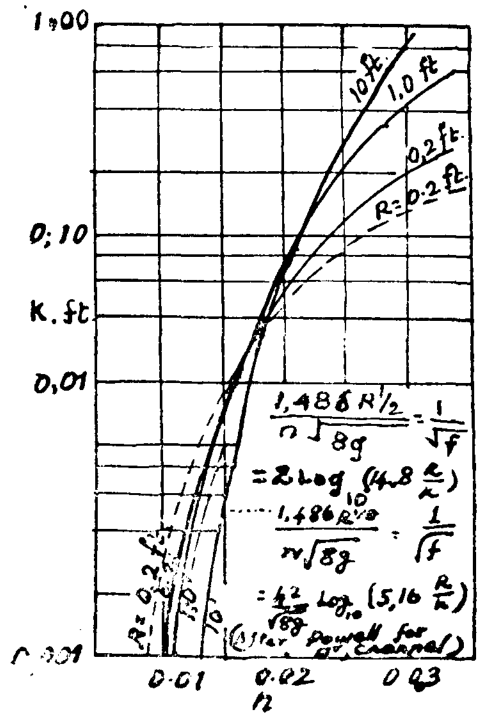
(*h*) Daugherty and Ingersoll[11] arrived at the relation in the following way : Writing Eq. 18 as

$$\frac{1}{\sqrt{f}} = 2 \log_{10} \frac{D}{k} + 1.14$$

$$= 2 \log\left(14.8 \frac{R}{k}\right) \quad .. \quad (39)$$

Comparing Eq. 37 and Eq. 39, the desired relationship is represented in graphical form as in Fig. 3.

Powell[12] found that Eq. 39 did not fit experimental data taken by himself and others. He proposed an equation which reduced to the form of Eq. 39 is shown in Fig. 3, plotted as a dashed line for a value of R=0·2.



Conclusion—

From the above studies which include 5 relations between n and k expressed in mathematical fashion and 3 graphical relations, one salient feature is at once apparent. A threefold increase in n (as between the probable value of 0·01 to 0·03) represents something between a hundred and thousandfold increase in k. That is why the value of k can be in error by 100 per cent or so without causing an error of more than a few per cent in velocity or rate of discharge.

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APPLICATION OF CRITICAL PATH METHODS IN THE CONSTRUCTION OF BUILDINGS IN PUBLIC WORKS DEPARTMENT.

BY

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While inviting tenders for the construction of buildings, the Department stipulates the programme of construction in terms of the percentage of the contract amount and the different periods of contract. The same percentage of contract and the periods of contract are entered in the agreement and the contractor should adhere to the programme.

Bar Chart.—In actual execution, to keep conrtol on the programming and planning of different items of work involved in the construction, a detailed programme is prepared on the bar chart. In the bar chart, the items of work with their quantities and the periods when they should be started and completed are shown. The progress made in each period is noted under the programme for each item. From the bar chart, it may be noted if the progress is made as per the programme. This is the method generally accepted for programming building works.

A TYPICAL BAR CHART IS GIVEN BELOW.

Sl. No.	Item of work.	Programme in days.			
		30	30	30	30
1	Foundation and basement.				
2	Shell in Ground Floor.				
3	Shell in First Floor.				
4	Finishing items in Ground Floor.				
5	Finishing items in First Floor.				

The bar chart appears to be simple to understand and to interpret but this is misleading because it does not show the sequential relationships and the interdependence of various items of work. The engineer-in-charge must memorise the inter-relation and interdependence of various items of work. On a complex project, the bar chart is of very little use as a method for planning and control because it is not possible to memorise and keep constantly under review all the complicated inter-relationships between the many different items of work.

The bar chart is also a tedious process and it does not indicate the period precisely on which the items should be completed. It will not also be apparent which are the items that should be completed as per schedule and the items where there could be leeway in its construction. The programming on Critical Path Methods does not have the above disadvantages.

Critical Path Methods.—The Critical Path Methods were introduced in the building construction practice in the U.S.A. in the year 1960 and in U.K. in the year 1962. The Critical Path Method consists of representing all items of work involved in the construction in Schematic diagram or net work. The Schematic diagram or net work depicts the sequence and interrelation of all the items of work in the construction. It is a mathematical method of planning the different items of work. If the time required to execute each item of work is known, the minimum time required to complete the building can be calculated through the Critical Path of the net work.

The net work can be prepared showing the sequence and interdependence of each item of work or groups of items of work. To begin with, it will be easier to draw the simple net work between groups of items of work and then to incorporate the details. To draw the net work, the building should be broken down in

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different items of work called "activity" and this can be taken from the abstract of the estimate or the tender schedule of the work.

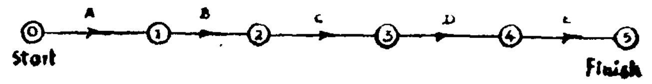
In the building construction, the items of work such as earth work, concreting foundations, masonry in foundation and basement, damp proof course and filling in basement may be grouped in one activity called 'Foundation and basement'. The masonry in Ground Floor, lintels, sun shades, laying concrete for floor slabs and beams may be grouped in one activity called 'Shell in Ground Floor'. Similar items in First Floor may be grouped in one activity called 'Shell in First Floor'. Finishing items such as laying concrete for the flooring, finishing plastering, fixing doors and windows, and painting may be grouped in one activity called 'Finishing items in Ground Floor'. Similar items of work in First Floor may be grouped in one activity called 'Finishing items in First Floor'. The construction of a building with two floors can be broken down in the above simple activities.

The table below shows the break-down of a two floor building in different activities and the time required to complete each activity in the number of days.

Activity.	Alphabet representing the activity.	Time required in days to complete the activity.
(1)	(2)	(3)
1. Foundation and basement	A	20
2. Shell in Ground Floor	B	30
3. Shell in First Floor	C	30
4. Finishing items in Ground Floor	D	20
5. Finishing items in First Floor	E	20

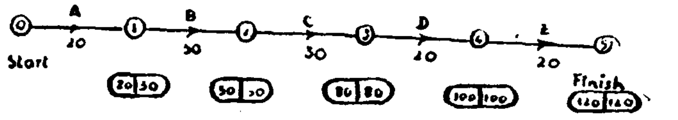
In the net work, each activity will be represented by an alphabet. In the above tables, A, B, C, D and E represent the activities.

Drawing of net work.—It may be programmed to take up one activity, complete that activity and then take up the next activity. In the above, it may be programmed to complete the foundations and basement before taking up shell in Ground Floor and similarly for other items. In this case the net work will be as follows :—



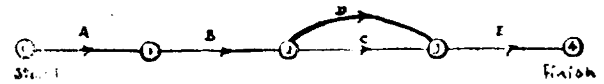
The net work shows that activity A should precede activity B, activity B should precede activity C, activity C should precede activity D and so on for other activities. The activity is represented by a separate line or arrow. The length of the arrow has no significance. It is convenient to have the horizontal projection of the arrows moving from left to right. The beginning activity should be drawn at the left end of the sheet and the arrows representing the activities should be positioned relative to one another and the net work should be built up progressively until the completing activity is reached. In the net work, the end of an activity is called an event and the events are shown by numerical number at the end of the activities.

Time required to complete the building.—The time required to complete the building can be determined from the net work if the duration of each activity, that is, the time (days or weeks) required for the completion of each activity is known. In the above case, the time required to complete each activity is shown in the table. In the net work the duration of the activity is entered as a number below the arrow while the description of the activity is written as an alphabet above the arrow.

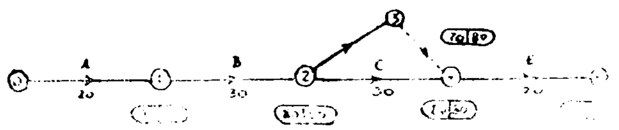


Proceeding through the events in a numerical order, from the start, simple addition of the duration will give the earliest possible time at which all the activities entering the event can be completed. This is the earliest finish time (EFT) for the event. The EFT for each event is recorded in the left side of the Oval "Time Box" adjacent to that event. After proceeding the net work, the EFT of the last event is derived. This is the earliest possible time in which the building can be completed. *This is the sum of the durations of the longest time path through the net work from start to finish.* In the above case, the earliest finishing time EFT of event No. 1 is 20 days; No. 2 is 50 days; No. 3 is 80 days; No. 4 is 100 days and No. 5 is 120 days.

To speed up construction, it will be logical to programme the finishing in Ground Floor (activity D) simultaneously with the shell in First Floor (activity C). That is the activities C and D can be concurrent. In this case, net work will be as follows:



In the net work, the concurrent activities are represented separately connected with a dummy arrow for unique identification of the event and to note the numbers in the "timing box" separately. The dummy arrow has zero duration. Hence the net work should be drawn as follows:—



By programming the activities C and D concurrently it may be seen that the earliest finish time (EFT) is reduced to 100 days. For the event 3, the end of activity D, these are two timings 70/80 in the time box. 70 is the earliest finish time (EFT) and 80 is the latest finish time (LFT) which is obtained working backwards by deducting the duration from the event 4.

The difference between EFT and LFT is the leeway available for delays and is called float. For some events, the same figure may appear in both sides of the box, indicating the same time for the latest finish time and the earliest finish time. In these cases there will be no float at all. These are the critical events that must be completed on the schedule if the building is to be finished in the minimum total time. *The path joining these critical events is therefore the critical path for the net work.*

Example :—

The application of the critical path method in the programme of the construction of a Hostel building for 102 students is indicated below.

The following particulars are extracted from the abstract of the estimate :—

Serial number and Particulars.		*Quantity.*
1	Earthwork	33,000 c.ft.
2	Concrete in Foundation	6,500 c.ft
3	Brickwork in Foundation and basement.	15,500 c.ft
4	R.C.C. Column footings	475 c.ft.
5	Damp proof course	2,150 s.ft.
6	Filling with excavated earth	28,300 c.ft.
7	Brickwork in Ground Floor	15,200 c.ft.
8	Brickwork in First Floor	10,300 c.ft.
9	Lintels in Ground Floor	400 c.ft.
	Lintels in First Floor	242 c.ft.
10	Tee or Ell beams in G.F.	721 c.ft.
	Tee or Ell beams in F.F.	501 c.ft.
11	R.C. beam in G.F.	612 c.ft.
	R.C. beam in F.F.	409 c.ft.
12	Bed blocks G.F.	22 c.ft.
	Bed blocks F.F.	16 c.ft.
13	R.C. Columns G.F.	722 c.ft.
	R.C. Columns F.F.	228 c.ft.
14	Doors	100 nos.
15	Windows	163 nos.
16	Ventilators	10 nos.
17	Louvers	33 nos.
18	Jali	328 s.ft.
19	Weld mesh screen	1,640 s.ft.
20	Flooring concrete	16,100 s.ft.
21	Plastering G.F.	33,650 s.ft.
	Plastering F.F.	15,400 s.ft.
	Plastering S.F.	8,150 s.ft.
22	Roof slab	16,710 s.ft.
23	Weathering course	16,710 s.ft.
24	Floor slab	18,600 s.ft.
25	Painting G.F.	7,750 s.ft.
	Painting F.F.	4,600 s.ft.
26	Partition walls G.F.	4,420 s.ft.
	Partition walls F.F.	1,520 s.ft.

Serial number and Particulars.		*Quantity.*
	airs	L. S.
	n water pipes	L. S.
	[illegible]	L. S.
	Canopy	L. S.
31	Electrical installation	L. S.
32	Providing built in cupboards	L. S.
33	Fixing cooking range	L. S.
34	Fixing shelves	L. S.
35	Fixing collapsible gates	L. S.
36	Providing Mosaic	L. S.
37	Expansion and Vertical Tower	L. S.
38	Pavement Drains	L. S.

The abstract of the estimate is treated as the break down of the building in different activities. From this, the construction logic showing the sequence of activities with their duration is tabulated below—

Serial number and description of activity. (1)	*Alphabet representing the activity.* (2)	*Duration in days* (3)
1 Earthwork	A	20
2 Foundation concrete	B	10
3 Column footings	C	2
4 Brickwork	D	24
5 Damp proof course	E	4
6 Filling in basement	F	2
7 Brick work in Ground Floor upto Lintels	G	24
8 Lintels canopy, sunshades	H	12
9 Brick work above Lintels	J	8
10 R.C. beams, slabs	J	40
11 Brick work in First Floor up to Lintels	K	16
12 Lintels, Sunshades in First Floor	L	10
13 Brick work above Lintels in F.F.	M	6
14 Columns in First Floor	N	4
15 R.C beams and Slabs	O	40
16 Plastering in Ground Floor	P	20
17 Flooring concrete	Q	14
18 Partition Walls	R	6
19 Floor finish	S	16
20 Fixing Doors, Windows, Louvers, Ventilation, etc.	T	16
21 Fixing Weld mesh screen	U	6
22 Weathering Course	V	16
23 Pavement Drain	W	14
24 Fixing Rain water pipes	X	6
25 Expansion Joints	Y	4
26 Partition Walls	Z	4

Serial number and description of activity. (1)	*Alphabet representing the activity.* (2)	*Duration in days.* (3)
27 Fixing Doors and windows, Louvers Ventilation in First floor.	A1	—
28 Plastering in First Floor	B1	10
29 Finishing Floor in F.F.	C1	6
30 Fixing weld mesh screen in First floor	D1	4
31 Electrical installation	E1	30
32 Fixing collapsible gates	G1	4
33 Cup-boards, shelves, cooking range	F1	14
34 Painting and polishing	H1	8

The net work is drawn for the above activity and it is determined from the net work that the total minimum time required to complete the building is 290 days. The critical path joining the critical events is shown in thick line.

It is assumed in drawing the net work that all the materials, labour, doors, windows, jali required for the work are available. If there is difficulty in the availability of the above items, it will increase the duration of the corresponding activity and the sequence of the activity. In developing the net work, it is usual to assume that all materials and labour are available and then to alter the net work to suit the limitations.

Points to be observed in drawing the net work:—

The first step in the preparation of the net work is to divide the building into different activities. The activities are arranged by listing them in columns with their duration. The specific ordering of the activities should be done in conformity with the construction logic which can be decided by an answer to the three simple questions:

(1) What activity must be completed before the activity being considered can begin?

(2) What activities can proceed simultaneously with this activity?

(3) What activities cannot start until this activity is completed?

The questions will decide the relative position of the activity in the net work.

It is easier to start with rough net work incorporating finer details successively than to attempt a detailed net work at the outset.

While developing the net work, the following principles should be followed :

(1) The order of numbering is such that the number at the head of the arrow is always greater than the number at the tail.

(2) Each node should represent correctly the complete relation between all activities entering and leaving.

(3) All activities entering a node should have identical followers and all activities leaving a node should have identical predecessors.

(4) Overlapping operations are prohibited. If they occur, they must be broken down into two or more activities representing these portions of the operation to be completed before later portions are commenced.

Suggestion—

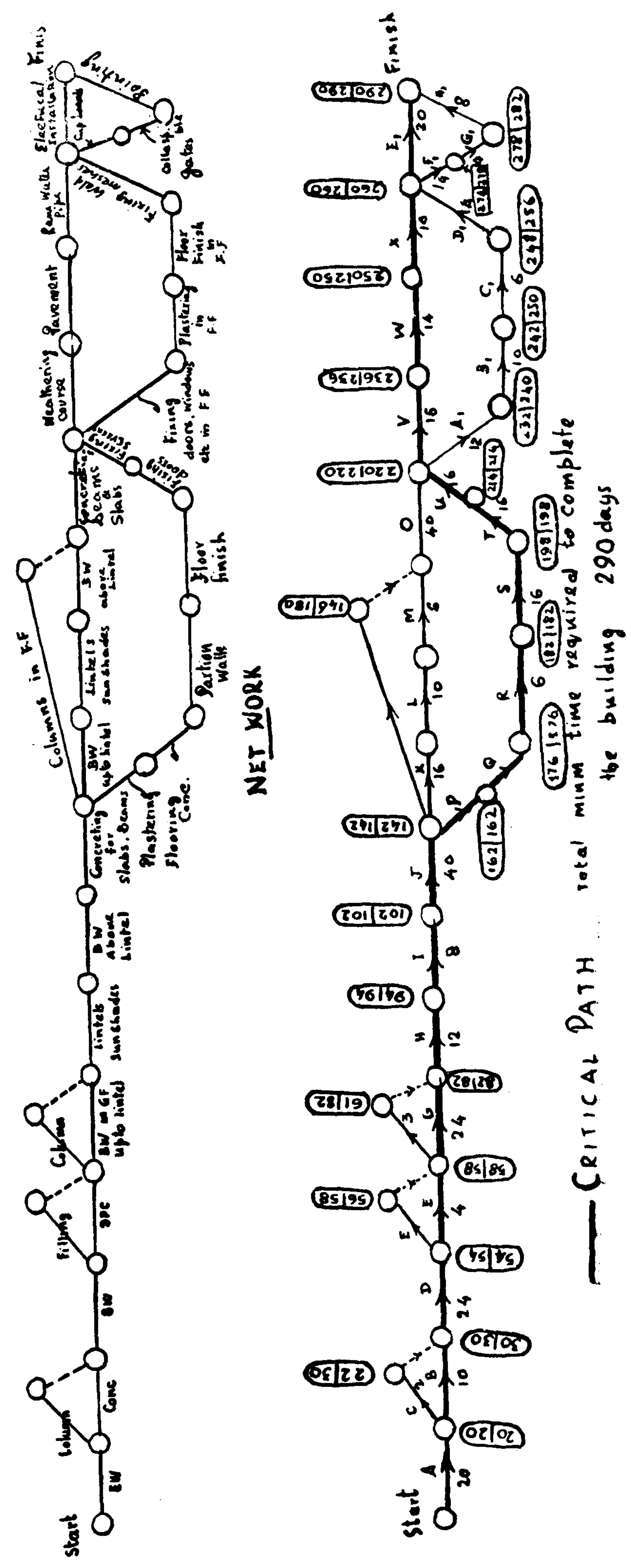
For adopting the programming of building work on the Critical Path Methods it is suggested that a sheet may be attached to the tender schedule containing a tabular column showing 'the activities' (different items of work) and a column to indicate the duration of the activity. The rough net work can also be drawn up on the sheet. The tenderer, considering his resources, control over labour and facilities, can be asked to fill in the duration of each activity required by him to complete it. The department on receipt of the tenders, can fill in the time box of the net work with the duration submitted by the tender, determine the CriticalPath and compute the total minimum time required to complete the building. This will be more rational and binding on the tenderer than the existing practice, chalked out without any basic data.

Reference :—

(1) Critical path Methods in Construction Practice—James M. Antill and Ronald W. Wood Head.

(2) The Critical Path method of Programming Civil Engineering Works—P.D. Arthur (Indian Concrete Journal—October 1966).

(3) The application of the Critical path method in construction management.—V. G. Gokhale (Indian Concrete Journal—November 1966).



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NEWS FROM ABROAD.

HIGHEST CHIMNEY IN THE OFFING.

Appalachian Power Co. last week broke ground for its $200-million, 1·6 million-kw Mitchell power plant at Moundsville, W.Va., and announced that it will have a 1,200 ft. high concrete chimney, setting a world record. The tallest so far are two 1,000 footers to be built for the Conemaugh plant in Pennsylvania (ENR 1/19 p. 21).

Mitchell's boilers will consume 4·4 million tons of coal annually to provide steam for a pair of 800,00 kw turbine generators. Although the plant site is on the banks of the Ohio River, only makeup water will be withdrawn. Cooling water will recirculate through two 370 ft. high concrete cooling towers.

The Mitchell plant is one of the largest single projects in a $1·37 billion, five year expansion of the American Electric Power Co.'s system, of which Appalachian Power is a subsidiary. AEP also is building a $200 million, 765 kv transmission network.

300 TON MASHER CLEARS RESERVOIR SITE.

It may look like a World War I Whippet tank, but this goliath weighs 300 tons and would make a very thin bale of scrap out of an old Whippet in short order.

Called a crawler-crusher, the 22 ft. high, 58 ft. long machine is prowling a forest in central Florida, knocking down cypress trees up to 6 ft. in diameter, for Gregg, Gibson and Gregg, Inc., the Leesburg, Fla., contractor that has a $4 million contract to clear the reservoir behind Rodman Dam, part of the Cross-Florida barge canal (ENR 1/27/66 P. 86).

A pair of 270 hp. diesel engines drive the rig's two 11½ ft. wide crawler tracks as it mauls down everything in sight, moving at about ½ mph. Although the machine appears to be nothing but a huge set of tracks with an operator's cab on top, its body is comprised of two steel tank sections complete with ship's bulkheads and watertight doors, making the mammoth landclearer amphibious. Except for very fast-flowing, deep rivers, water does not stop the behemoth, which draws 7 ft. 10 in. of water. Grousers, 5 in. high, on its tracks provide propulsion when running in water.

Up front, a tubular steel push bar filled with concrete takes on the big trees initially. The contractor claims the rig has pushed over eight 24 in. dia. cypress trees simultaneously as though they were nothing more than heavy ground cover. Because of its great weight, the crusher not only knocks down everything but mashes it into the relatively soft ground so that even the biggest trees will not float when the reservoir fills. It leaves behind a shallow depression 24 ft. wide.

The tow-part body, which might better be called a hull, is compartmentalized with ballast tanks below and engine rooms, fuel tanks and storage areas on the upper levels. Two men, an operator and an engineer, ride in the control house, which, along with the engine room air supply system, make up all the superstructure.

Without fuel or water ballast the unit weights 204 tons and can be disassembled for over-the road transport. Fully loaded it weighs 306 tons and will level about 1 acre an hour.

—E.N.R., June 1967.

புதுயுகப் பாசனம்

" வரப்புயர நீருயரும், நீருயர நெல்லுயரும்,
நெல்லுயர குடியுயரும், குடியுயரக் கோலுயரும்."

—ஔவையார்.

தமிழகத்தில் நீர்ப்பாசனம்-III *

ஓர் ஆய்வுரை.

மொழிபெயர்ப்பு—திரு. வெ. இளங்கோவன், B.E., Assoc. I.S.E., ‡

4. பாசனத் திட்டங்களின் எதிர்காலம்.—ஏறக் குறைய தமிழகத்திலுள்ள ஒவ்வொரு நதியிலும் அமைக்கப்பட்டுள்ள நீர்த்தேக்கங்களின் பயனாக, நீரை எந்த அளவிற்குப் பயன்படுத்த முடியுமோ அந்த அளவிற்குப் பயன்படுத்தப்பட்டு வருகிறது. எனவே, புதிய பெருந் திட்டங்களை ஏற்படுத்தும் வாய்ப்பு மிகக் குறைவாகவே உள்ளது. தமிழகம் முழுவதிலும் 15 செ.மீ. (6 அங்.) ஆழத்திற்கு நீரைத் தேக்கி வைப்பதற்கு வேண்டிய 20,000 மிலியன் கன மீட்டர் (00,000 கன அடி) நீரைத் தமிழகத்திலுள்ள ஆறுகள் எடுத்துச் செல்கின்றன என்று மதிப்பிடப்பட்டுள்ளது குறிப்பிடத்தக்கது. இந்த அளவு நீரில் ஏறக்குறைய 18 000 மிலி. க.மீ. (650,000 மிலி. க.அ.) ஏற்கனவே பாசனத்திற்காகப் பயன்படுத்தப்பட்டு வருகிறது. மீதமுள்ள நீர் எப்பொழுதாவது வரும் தவிர்க்க முடியாத வெள்ளப்பெருக்கினால் ஆகும். அதை முழுவதும் பயன்படுத்துவதென்பது கடினமான செயலாகும்.

நான்காவது 5 ஆண்டுத் திட்டத்தில் உத்தேசிக்கப்பட்டுள்ளத் திட்டங்கள்.—இருப்பினும் தல திட்டங்களினால் (Local schemes) கிடைக்கப்பெறும் மிகுதி நீரைக்கொண்டு பிற்பட்ட பகுதிகளுக்குப் பயன்படச் செய்யும் முயற்சிகள் இன்னும் உள. இப்பொழுது இயங்கிவரும் திட்டங்கள், மேலும் புதிதாக விரிவுபடுத்தப்பட்டுள்ள திட்ட வேலைகள் ஆகியவற்றிற்குத் தேவையான நீரளவுபோக மீதமுள்ள குறைந்த நீரைக்கொண்டுதான் மேற்குறிப்பிட்ட தல திட்டங்களுக்குத் தேவையான நீர் தரப்படும். இதனால் இத்திட்டங்கள் எல்லா ஆண்டுகளிலும் பலனைத் தராவிடினும், நிலைமைகள் அனுமதிக்கும் ஆண்டுகளில் இம்மாநிலத்தின் உணவு உற்பத்தியைப் பெருக்க உதவியாயிருக்கும். இதைக் கருத்தில் கொண்டு நான்காவது ஐந்தாண்டுத் திட்டத்தில் சில திட்டங்களைச் செயல் படுத்த முயற்சி எடுக்கப்படுகிறது. பரப்பலாறு, சின்னாறு, பாலாறு, பொரந்தலாறு, மருதையாறு முதலிய திட்டங்கள் மூலம் ஆங்காங்கேயுள்ள ஓடைகளின் குறுக்கே நீர்த் தேக்கங்களை ஏற்படுத்தி அருகேயுள்ள நிலங்களுக்குப் பயன்படச் செய்ய ஏற்பாடு செய்யலாம். 46 மீ. (150 அடி) உயரமுத்த மட்டத்திற்கு (Head) பவானியாற்று நீரை ஏற்றி கோவை மாவட்டத்தில் அவனாசி தாலுகாவிலுள்ள வரண்ட நிலப் பகுதிக்கு நீர் அளிக்கும் திட்டமும் அரசாங்கத்தின் கவனத்தைக் கவர்ந்துள்ளது.

இன்னும் நீர்ப்பாசன வசதிகளைப் பெருக்கவேண்டுமானால் அவைகளைப் பழைய பாசன முறைகளைப் புதிப்பிப்பதாலேயே (Modernisation) அடையமுடியும். அப்படிச் செய்கையில் பல்லாண்டு காலமாகவே உபயோகத்திலிருந்து வரும் சிறு வாய்க்கால்களை தனித்தனியாகத் திருத்தி அமைக்க முயற்சி செய்யலாம். ஆனால், காவிரி டெல்டா, பெரியாறு போன்ற பெருந்திட்டங்களைப் புதுப்பிக்கக் கூட்டு முயற்சியே தேவையாம்.

1930–35-ல் காவிரி டெல்டா பகுதியில் இத்தகைய முயற்சிகள் செய்யப்பட்டன. ஆனால் அவைகள் மதகுகளைத் திருத்தி அமைப்பது, மதகுப் பலகைகளை வைப்பது, ஆற்றிலிருந்து வெளியே எடுக்கும் நீரைக் கட்டுப்படுத்துவது ஆகியவைகளோடு நின்றுவிட்டது. இருப்பினும், கால்வாய்களைச் செப்பனிடுதல், நீர்க்கசிவு முதலிய இழப்பைக் குறைவு படுத்தல், நீரைத் தக்க அளவு விநியோகித்தல் முதலியவற்றின் மூலம் நீரை நன் முறையில் சேமிக்க நல்ல வாய்ப்புகள் உள. குறிப்பிட்ட அளவு நீரைக் கொண்டு அதிக அளவு உற்பத்திக்கு வழி செய்யும் நல்ல நீர் விநியோக செயலாற்று முறை (Water Management) தேவை. 4-வது ஐந்தாண்டுத் திட்டத்தில் காவிரி டெல்டாவைப் புதுப்பிக்கும் திட்டமொன்று உத்தேசிக்கப்பட்டுள்ளது. 25,750 கி.மீ. (16,000 மைல்) நீளமுள்ள ஆறுகளும், கால்வாய்களும் செப்பனிடப்படும். இதன்படி ஆற்றுத் தலைப்புகளில் (Open Heads) மதகுகள் கட்டப்படும். விநியோகமுறைகள் கட்டுப்படுத்தப்படும். கசிவு இழப்பு அதிகமுள்ள இடங்களில் அவைகளைக் குறைக்க காங்கிரீட் பாவு வேலைகள் செய்யப்படும். நீரைச் சிக்கனமாக உபயோகப்படுத்தும் முறையை உழவர்கட்கு அறிவிக்கும் பொருட்டு ஆங்காங்கே செயல்விளக்கம் (Demonstration) அறிவிக்கப்படும் திட்டமும் மேற்கொள்ளப்படும்.

அதுபோன்றே 57,870 ஹெக்டார் (143,000 ஏக்கர்) நிலங்களை, அறுபது ஆண்டுகளுக்கும் மேலாகப் பாசனப்படுத்திவரும் பெரியாற்றுத் திட்டத்தின் கீழுள்ளக் கால்வாய்களும் நீரைச் சிக்கனப்படுத்துவதற்காகப் புதுப்பிக்கப்படும்.

பழைய முறையிலுள்ள நீர்பாயும் வழிகள் (alignments) முறைகள், திட்டங்கள், கடைபிடித்துவந்த மரபுகள் ஆகியவற்றால் ஏற்படுகின்ற இடர்பாடுகளைக் களைவது மிகக்கடினமே. ஆயினும், அதிக அளவிற்கு நல்ல பலன்கள் கிடைக்க வேண்டுமானால் உறுதியான முயற்சி தேவை.

*Translated from the article on "Irrigation in Madras State—A Review", prepared by Thiru T. V. Subramanian, Assistant Engineer, Office of the Chief Engineer for Irrigation.

‡ Junior Engineer, Ganging Division, P.W.D., Madras-5.

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இத்திட்டங்களனைத்தையும் செயல்படுத்தினாலும், மிக விரைவாகப் பெருகிவரும் மக்கட் தொகைக்கு வேண்டிய உணவுத்தேவையைப் பூர்த்தி செய்யமுடியாது. அருகிலுள்ள மாநிலங்களின் பெரிய நதிகளிலிருந்து உபரி நீரைப் பெருவதின் மூலமே நீர்ப்பாசனத்தை இன்னும் சற்று விரிவு படுத்தமுடியும். அப்படிப் பார்க்கும்பொழுது, கேரள மாநிலத்தின் மலைகளினூடே மேற்கு நோக்கிப் பாயும் ஆறுகளின் மிகுதி நீரைத் திருப்பிவிடுவது எளிதான செயலே. எனவே கேரள மாநிலத்திலிருந்து நீரைப்பெருகின்ற வழிகளும், முறைகளும் ஆராயப்பட்டு வருகின்றன. பரம்பிகுளம்-ஆளியாறு திட்டத்தின் கீழ் மேலும் அதிக நீரைத் திருப்பிவிடுவது மட்டுமின்றி, செயல்படுத்தக்கூடிய இன்னபிறத் திட்டங்களும் நான்காவது ஐந்தாண்டுத் திட்டத்தில் கவனிக்கப்படும்.

நான்காவது ஐந்தாண்டுத் திட்டத்தில் சேர்க்கப்பட்டுள்ளத் திட்டங்களை பிற்சேர்க்கை III-ல் காணலாம்.

5. தனிச் சிறு பாசனத் திட்டமும், தூர்வாரல்—நிலமீட்டுப் பணிகளும்.—உடனடியாகப் பலன்களைத் தரத்தக்க சிறு தலப்பணிகளே (Local Works) தனிச் சிறிய பாசனத் திட்டத்தின் (Special Minor Irrigation Programme) கீழ் செயல் படுத்தப்படும் திட்டங்கள். புதிய ஏரிகளை அமைத்தல், அணைக்கட்டுகளைக் கட்டுதல், விநியோகக் கால்வாய்களை வெட்டுதல், ஏற்கெனவேயுள்ள பாசன மூலங்களை (Irrigation Sources) சரி செய்து, நன்முறையிலமைத்தல் முதலிய பணிகள் சிறிய பாசனத் திட்டங்களுக்குட்பட்டது. அப்பணிகள் இரு செயலடையீடுகளின் (Agencies) கீழ் செய்யப்படுகின்றன. அதாவது வருவாய்த் துறைக் கழகத்தின் (உணவு உற்பத்தி) கீழுள்ள உணவு உற்பத்திப் பொறியியல் பணியாளர்களாலும் (Food Production Engineering Staff) தலைமைப் பொறியாளர் (நீர்ப்பாசனம்) அவர்களின்கீழ் செயல்படும் பொதுப்பணித் துறைப் பொறியாளர்களும் பணி புரிவர். உணவு உற்பத்தித் துறைப் பொறியாளர்கள், வருவாய்த் துறைக் கழகத்தின் கட்டுப்பாட்டிலுள்ள பாசன மூலங்களையும் (Irrigation Sources) பழைய நாட்களிலிருந்துவரும் ஜமீன் பகுதிகளையும் கவனித்துப் பண்புரிவர். பழைய ஜமீன் பகுதிகளிலுள்ள பாசனமூலங்களைச் சரிசெய்து, உணவு உற்பத்தித்துறைப் பணியாளர்கள் அவற்றை பொதுப் பணித் துறைக்கோ அல்லது வருவாய்த் துறைக் கழகத்திற்கோ அவ்வேலைகள் யாரின்கீழ் உள்ளதோ அவர்களுக்கு மேற்பார்வைக்காக அளிப்பர். அதன்படி பொதுப் பணித் துறையினர் அவர்களின் கீழ் உள்ள வேலைகளைக் கவனிப்பர்.

மூன்று ஐந்தாண்டுத் திட்டங்களிலும் அடைந்த பலன்கள் முதலியவைகள் கீழேயுள்ள அட்டவணையில் கொடுக்கப் பட்டுள்ளன.—

தனிச் சிறு பாசனத் திட்டம்.

(உணவு உற்பத்தித் துறையும், பொதுப் பணித் துறையும்.)

காலப்பகுதி.	செலவிடப்பட்ட தொகை (மிலியன் ரூபாயில்).	முடிக்கப்பட்ட பணிகளின் எண்ணிக்கை.	பலனடைந்த பகுதி. இடையே விடப்பட்டனவும் புதிய திட்டங்களும் ஹெக்டரில் / ஏக்கரில்.	நிரந்தரமாய்ப் பலனடைவன ஹெட்டரில் / ஏக்கரில்
(1)	(2)	(3)	(4)	(5)
முதல் ஐந்தாண்டுத் திட்டம்	29·00	2,458	12,950	174,015
இரண்டாவது ,, ...	47·35	2,250	32,000 75,919	430,000 ..
மூன்றுவது ,, ...	111·40	4,286	187,600 54,838	166,457
			135,402	411,005

தூர்வாரல்—நிலமீட்பு—மாநிலத்தின் மொத்த நீர் பாசனத்திற்கு வேண்டிய நீரில் 40 சதவிகிதத்தை 37,000 ஏரிகள் அளிக்கின்றன. எனவே, மாநில பொருளாதாரத்தில் இந்த ஏரிகள் மிக முக்கியபங்கை வகிக்கின்றன. விரைந்தோடும் காலத்தினால் அநேக ஏரிகள் படிப்படியாகவும் மெதுவாகவும் படியும் வண்டல் மூலம் பயனுள்ள கொள்ளளவை இழந்துவிட்டன. எனவே தூர்வாரல்—நிலமீட்புத் திட்டத்தினால் அவற்றை பழைய நிலைக்குப் புதுப்பிக்க வேண்டிய அவசியம் ஏற்பட்டுள்ளது. இத்திட்டத்தை மற்ற மாநிலங்களும் பின்பற்றுகின்றன. ஏரியின் அடியிலுள்ள எல்லா வண்டலையும் நீக்குவதே இதன்முக்கிய நோக்கமாக முன்பு இருந்தது. ஆனால் காலப்போக்கில் அவற்றை நிறைவேற்றுவதில் பல இன்னல்களைக் காணவேண்டியிருந்தது. எனவே இத்திட்டங்கள் திருத்தி அமைக்கப்பட்டன.

திருத்தியமைக்கப்பட்ட தூர்வாரல்-நிலமீட்புத் திட்டத்தினால் ஏரிகளின் இழந்த கொள்ளளவை ஓரளவாவது மீட்பது அல்லது ஏரியின் முழு உயர மட்டத்தை (F.T.L.) எங்கெங்கு உயர்த்தவேண்டுமோ அங்கெல்லாம் உயர்த்தி, தூர்வாருவது என்பது போன்ற பணிகள் மேற்கொள்ளப்படும். தற்பொழுது

உத்தேசிக்கப்பட்டுள்ள திட்டம் என்னவென்றல் மீட்கப்பட்ட புஞ்சை நிலத்திலும், புரம்போக்கு நிலத்திலும் தழை உரச் செடிகள் பயிரிடப்படும். மேலும் நஞ்சை பட்டா நிலங்களில் நஞ்சைப் பயிர்கள் விளைவிக்கப்படும், மண்வாரும் இயந்திரங்களின் மூலமும், முடிந்த அளவு மனித உழைப்பைக் கொண்டும் ஏரிகளில் தூர்வாரப்படும்.

முதலிரண்டு ஐந்தாண்டுத் திட்டங்களில் அடைந்த பலன்கள் கீழே தரப்பட்டுள்ளன.

திட்டம்.	செலவிடப்பட்டத் தொகை (மிலியன் ரூபாயில்).	முடிக்கப்பட்ட வேலைகள்.	பயனடைந்த நிலப்பரப்பு. அணைகரை மீட்பு ஹெக்டர்/ஏக்கர்.	இடைவெளி ஹெக்டர்/ ஏக்கர்.
(1)	(2)	(3)	(4)	(5)
முதல் ..	4·17	38	..	918/2268
இரண்டாவது	8·00	50	304/750	708/1750
மூன்றாவது (நோக்கம்) ..	7·74	38	332/821	635/1570
நான்காவது .. (1961–65) இடைத்த பலன்.				

பிற்சேர்க்கை III.

மேற் கொள்ள இருக்கும் 4-ஆவது ஐந்தாண்டுத் திட்டப்பணிகள்.

எண்.	திட்டத்தின் பெயர்.	நதி.	பயன்பெறும் மாவட்டம்.	மதிப்பீட்டுத் தொகை (லட்சம் ரூபாயில்).	பயனடையும் ஆயக்கட்டு. புதிய ஆயக் கட்டு ஆயிரம் ஹெக்டாரில்/ ஏக்கரில்.	நிரந்தரமாக்கப் பட்ட ஆயக்கட்டு ஆயிரம் ஹெக்டரில்/ ஏக்கரில்.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	மணிமுக்தா நதித் திட்டம்	மணிமுக்தா நதி.	தென்னுற்காடு.	91·00	1·616/4·000	0·101/0·250
2	இராம நதித் திட்டம் ..	இராம நதி.	திருநெல்வேலி.	87·00	0·201/0·500	1·213/2·998
3	கடனா நதித் திட்டம் ..	கடனா நதி.	திருநெல்வேலி.	158 00	0·405/1·000	2·878/7·112
4	வைகைக் கால்வாய்களைப் புதுப்பித்தல்.	வைகை நதி.	மதுரை, இராம நாதபுரம்.	395·00	5·625/13·900	38·794/95·862
5	தஞ்சைக் கால்வாய்களை நவீனப்படுத்தல்.	காவிரி நதி.	தஞ்சை	1,150·00	14·164/35·000	43·706/108·000
6	பெரியாற்றுப் பாசனத் திற்கு முன்னேற்ற வேலைகள் செய்தல்.	பெரியாறு.	மதுரை	400·00	....	....
7	அவனுசி வட்டத்திற்குப் பாசனவசதி செய்தல்.	பவானி ஆறு.	கோயமுத்தூர்	560·00	43·706/108·00	....
8	பரப்பலாற்றுத் திட்டம் ..	நங்காஞ்சி ஆறு.	மதுரை	33·57	0·405/1·000	0·535/1·323
9	பாலாறு-பொரந்தலாறு திட்டம்.	சன்முகா நதி.	மதுரை	256 00	2·076/4·981	4·047/10·000
10	சின்னாற்றுத் திட்டம் ..	சின்னாறு.	கோயமுத்தூர்	40·00	0·481/1·190	1·064/2 630
11	மருதையாற்றுத் திட்டம்	மருதையாறு.	திருச்சி	325·00	5·488/13·560	3·011/7·440
	மேற்கு நோக்கிப் பாயும் ஆறுகளைத் திருப்பிவிடல்—					
12	பரம்பிக்குளம்-ஆளியாறு திட்டம். (இரண்டாம் கட்டம்).	....	கோயமுத்தூர்	1,300·00	32·375/80·000	....
13	இடுக்கிப் பெரியாற்றுத் திட்டம்.	....	இராமநாதபுரம்	1,300·00	32·375/80.000	....
14	கல்லாறு திட்டம் ..	....	மதுரை..	155·00	7·284/18·000	....
15	அச்சன் கோயில் புனலாறு திட்டம்.	....	திருநெல்வேலி	500·00	16·070/15·000	....
				6,750·57	152·214/376·131	95·349/235·615

—[முற்றும்]—

THE NEW IRRIGATION ERA

Journal of the Irrigation Branch of the P.W.D. of Madras State

INDEX FOR VOLUME X.

(*N.B.*—Figures indicate page numbers and those within parantheses, issue numbers.)

SCHEME FOR SUPPLY OF CAUVERY RIVER WATER TO MADRAS

HON'BLE MINISTER FOR WORKS ADDRESSING ON THE OCCASION

VOTE OF THANKS BY THE SUPERINTENDING ENGINEER (P.H.), MADRAS

(1)	(2)	(1)	(2)

INVITATION TO READERS

Kind attention of the Engineers of the Madras Public Works Department and readers of the journal elsewhere is drawn to the meagre response to repeated requests for contribution of articles to the journal.

May we once again emphasise the two-fold objective of the journal, viz., to promote the interest of engineers and aiding their specialised work by placing before them the many problems of technical importance and ways and means of solving them, and secondly, opportunities being scarce for engineers to express their own experience in solving practical problems of design and execution, we wish to add that this journal has been the only medium of expression for those in the field. It is earnestly desired that the Engineering fraternity will come forward in large numbers to utilise this medium to its best advantage by offering their valuable contribution.

The scope of the articles to the journal need not be rigorously confined to the subjects on irrigation alone. Articles which have a direct bearing on subjects given below are also welcome:—

(1) Technical aspects of foundation soils and foundation Engineering.

(2) Auxiliary works in the field of irrigation and other water-retaining structures and works.

(3) Practical solution to hydraulic problems in canals and distribution systems with reference to their maintenance.

(4) Problems of soil erosion and conservation and reclamation of land.

(5) Practical silt problems in rivers and reservoirs.

(6) Problems of sanitation and public health and possibilities of utilisation of sewage.

(7) Inland waterways and navigation, fishing schemes and protection of land against sea erosion, etc.

(8) Problems of drainage in deltaic tracts and entrance of rivers into the sea

(9) Run-off studies in catchments and evaporation studies in reservoirs.

(10) All allied subjects.

The Executive Engineers of the Madras Public Works Department are specially requested to encourage their Subordinate Officers to come forward with articles and enable the journal to flourish and serve the common cause of the Engineers of the State.

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