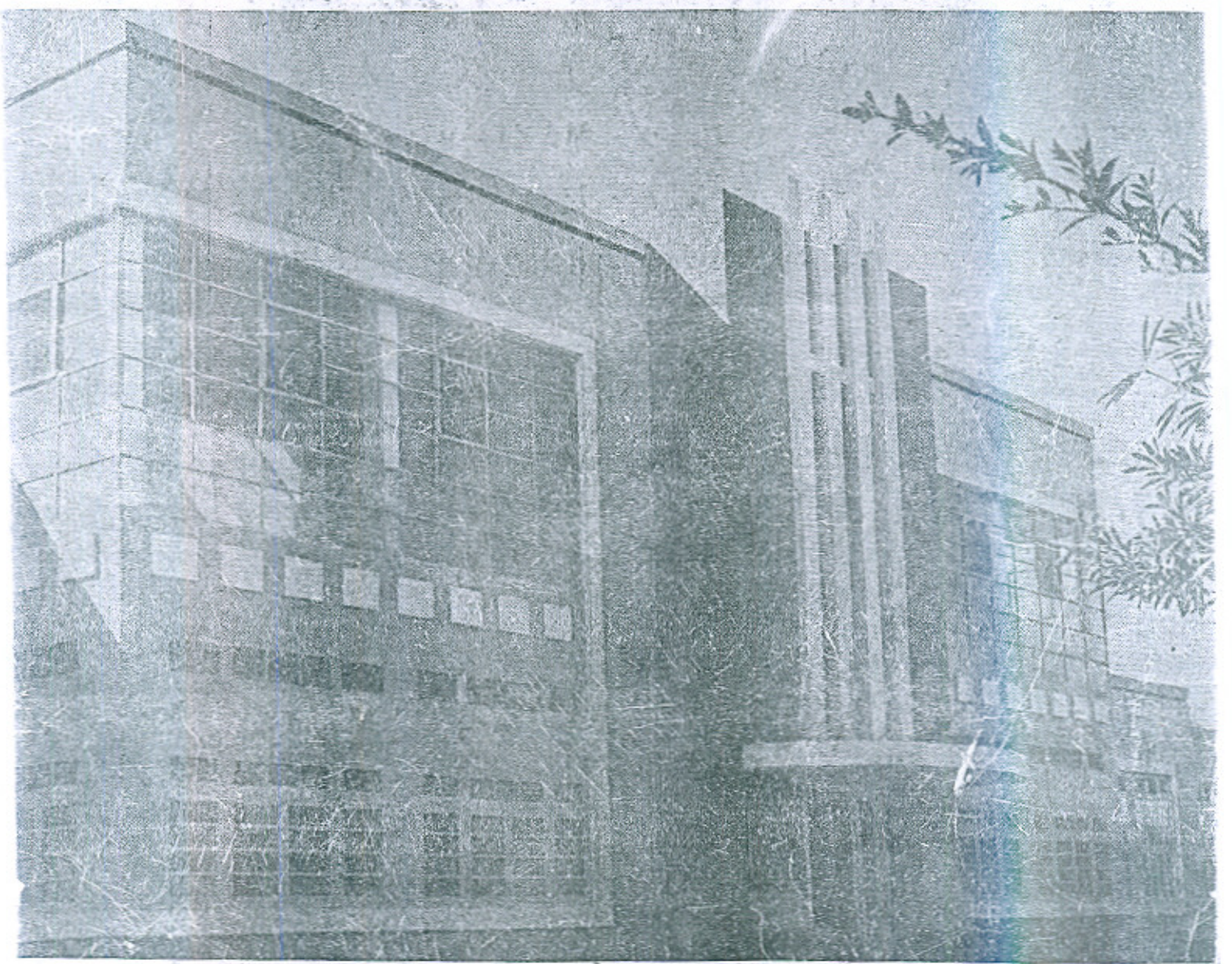
ENGINEERING NEWS

QUARTERLY JOURNAL OF THE WEST PAKISTAN ENGINEERING CONGRESS

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"For I remember stopping by the way
To watch a Potter thumping his wet clay:
And with its all-obliterated Tongue
It murmur'd—"Gently, Brother, gently, pray!"

"OMAR KHAYYAM"

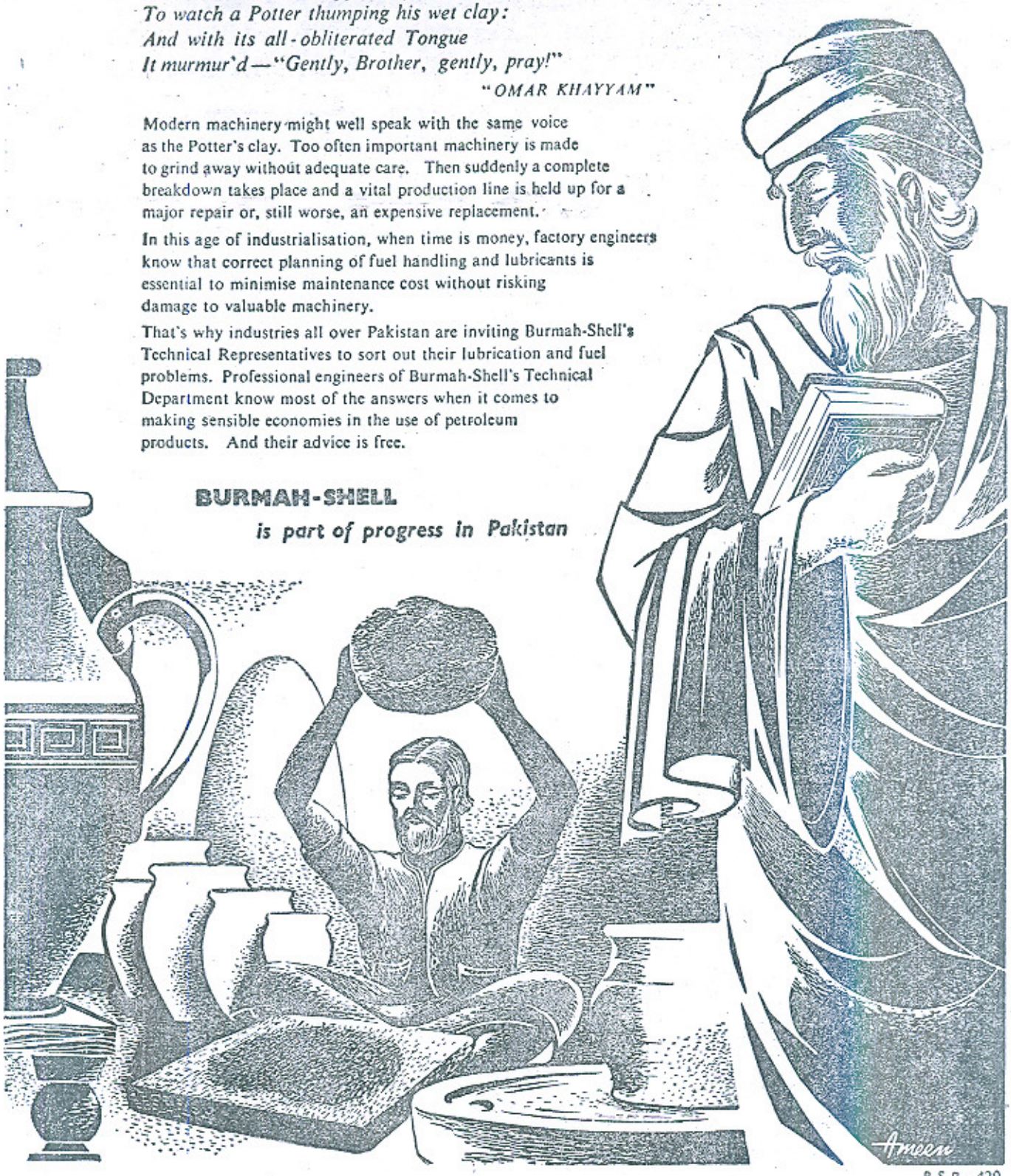
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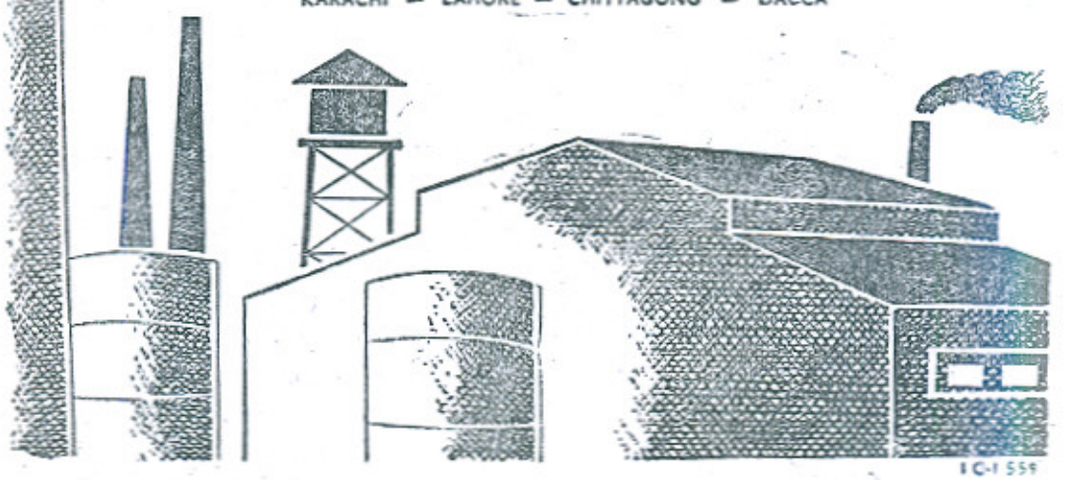
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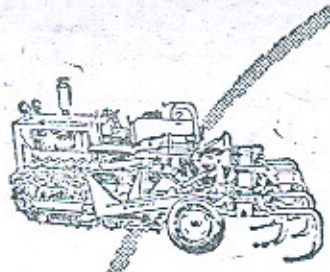
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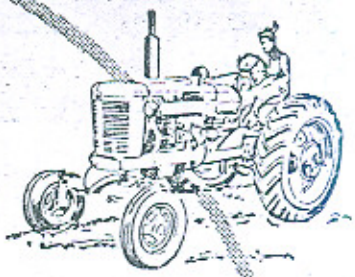
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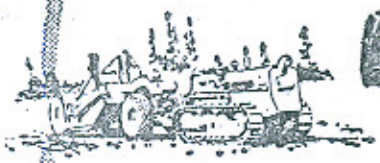
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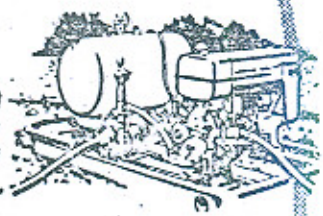




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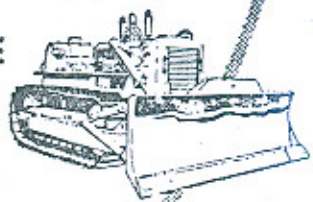


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Adsitos

A FEATURE ON THE WARSAK HYDRO-ELECTRIC PROJECT

SERIES—7

THE DESIGN OF MAIN DAM

By
Asrar Ahmad Qureshy*

THE dam consists of a straight gravity concrete structure 650 ft. long with a maximum base thikness of 211 ft. and the greatest vertical height of 235 ft. (lowest point in the dam foundation to the roadway).

The exposed rock formation in the original river bottom at site lies at an average elevation of 1055 feet above sea level and the roadway top of the dam is at elevation 1290. The crest of the nine 40-feet spillway openings is at elevation 1230. Nine electrically operated steel gates known as Tainter Gates are to be installed along the concrete spillway crests. The gates will be 40 feet high and will raise the reservoir level to 1270 when fully closed. A reinforced concrete roadway bridge spans the spillway. The roadway is 19 feet wide. There are no low level outlets.

The power house is nearly 90 ft. wide and 500 ft. long situated on the south bank of the river immediately downstream of the dam and adjacent to the spillway. There will be 6 generating units of 40,000 K.W. each served by an individual penstock which takes from the main power tunnel. The units consist of 55,000 h.p. vertical Francis Turbine directly coupled to the generators. The current generated will be carried through underground cable tunnels to step-up transformers in an outdoor switch-yard, located on the top of the power house.

*Executive Engineer, W.D.P.O.

DAM.

Structural Analysis :—

In order to test the adequacy of the final design of the dam for the various possible conditions of loading, it was necessary to make extensive analysis and studies of the stress and stress distribution, sliding factors and other factors such as earthquake effects.

The gravity analysis were made on the assumption that the full water load would be carried by gravity action and that the distribution of vertical stress from the upstream to the downstream face of the dam would be according to the straight line theory at all elevations. No twist action, which assumes that a part of the load is transferred to the abutments by twist action through a system of horizontal beam elements, is considered. The entire water load is assumed to be transferred vertically to the foundation by gravity action through a system of cantilever elements.

Basic Data :

The design of the cross-section was based on a reservoir surface elevation at 1285, the assumed maximum possible reservoir surface. With this loading the thickness of cross-section and slopes of upstream and downstream faces were proportioned so that the sliding factor, for the assumption of two-thirds uplift, would not exceed 0.65 in any part of the structure except where it was reinforced

against horizontal sliding or shear. The maximum earthquake was assumed to occur when the reservoir was filled upto the top of the tainter gates at elevation 1270.

The non-overflow or abutment sections have a base thickness of 145 ft at the assumed minimum level 1130 ft. In the spillway section, for purposes of analysis, a theoretical 0.70 downstream slope assumed between the end of the ogee crest curve to the foundation level, and the apron curve near the base of the section was ignored.

The plan, elevation and sections of the dam are shown in Fig. 1 and 2.

The principal items of specified structural data which controlled the final dam design are :—

(1) Elevation of top of dam 1290.

(2) Width of top of dam, in abutment section and spill way section is 19 ft.

(3) Location of spillway, main river channel.

(4) Total length of spillway, 360 feet (Clear waterway, excluding piers).

(5) Spillway control, nine 40 ft. × 40 ft. Tainter Gates.

(6) Elevation of permanent spillway crest 1230.

(7) Elevation of top of gates closed position, 1270.

(8) Alignment of vertical portions of upstream face, straight and continuous through spillway and both abutments sections.

Assumptions :

The following assumptions were made in the stress and stability analysis :—

(1) The transverse contraction joints in the dam are neither keyed nor grouted.

(2) All loads are carried by gravity action or weight of vertical parallel side cantilevers which receive no support from adjacent elements

(3) Unit vertical pressures or normal stresses and shear stresses on horizontal planes are assumed to vary linearly from upstream face to downstream face.

(4) Unit horizontal pressures or normal stresses on vertical planes and vertical shear stresses are assumed to vary linearly between sections.

Theoretically, the variation in shear stresses on both horizontal and vertical planes is of parabolic form but the approximation made in assuming a straight line variation is so small as not to affect the calculated stresses appreciably. The only case where this assumption is wrong is when both the upstream and downstream faces of the dam are vertical. As this case only occurs near the top of the bulkhead section, it does not influence the stresses computed here.

PLATE NO. 2.

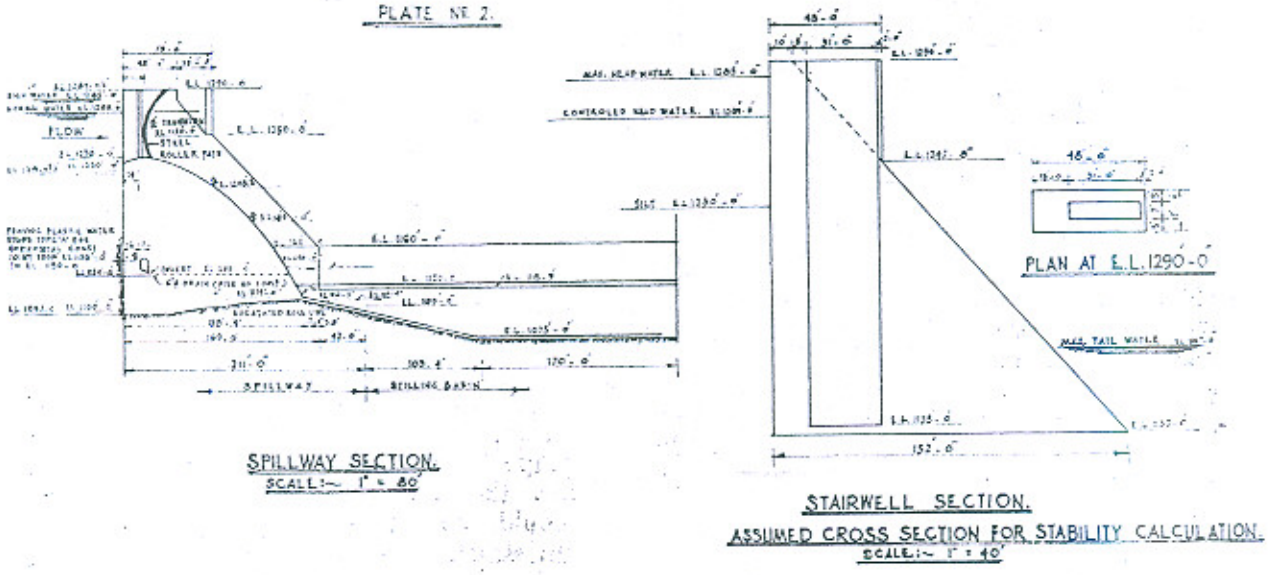
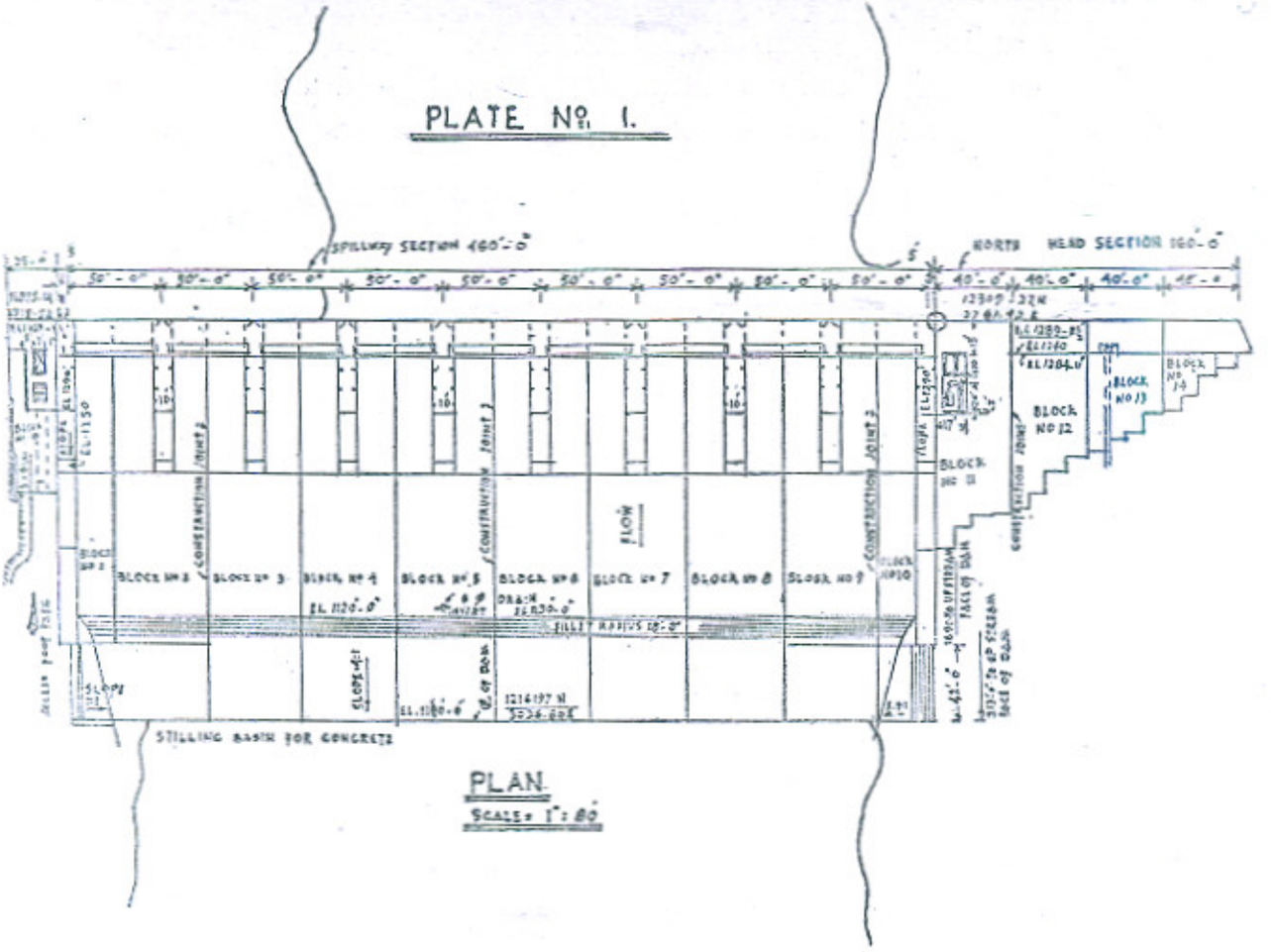
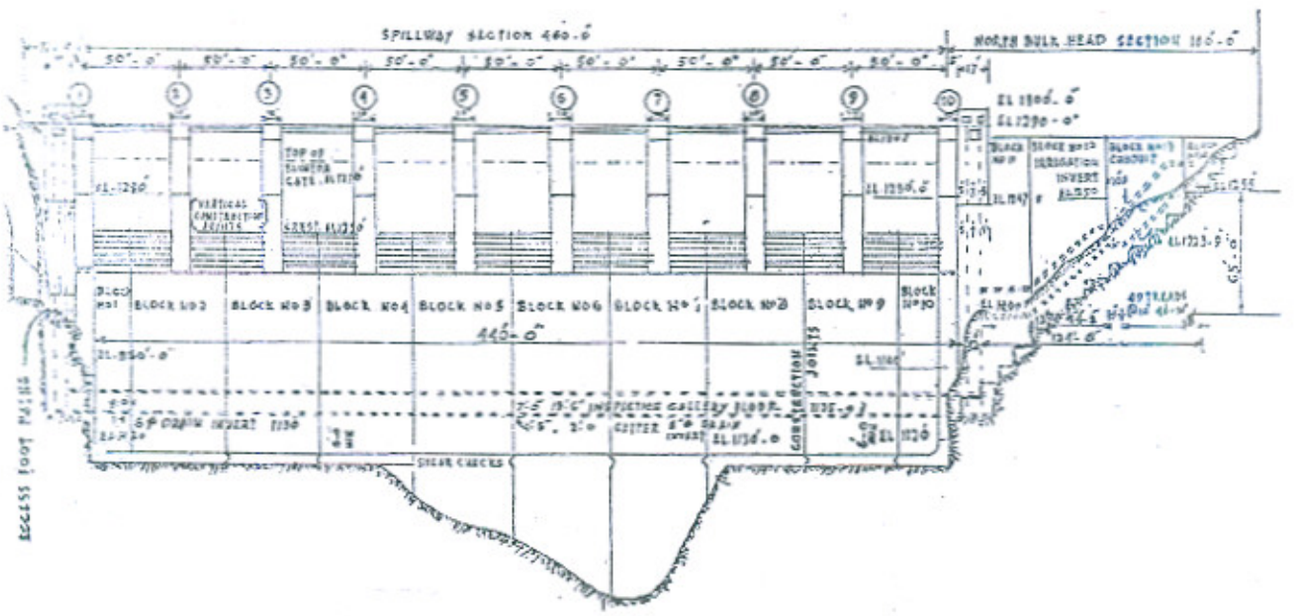


PLATE No. 1.



PLAN

SCALE:- 1" = 80'



DOWN STREAM ELEVATION

SCALE:- 1" = 80'

Constants used in the stability analysis are:—

(1) Unit weight of water = 62.41bs/c.ft.

(2) Unit weight of concrete = 145 lbs/c.ft.

(3) Modulus of elasticity of concrete in compression & tension lbs/sq. in. = 4,000,000

(4) Modulus of elasticity of concrete in shear lbs/sq in. = 1,667,000

(5) Dry wt. of silt = 90lbs/c. ft.

TABLE 'A'

Allowable Values

Values	4000 psi. used for the 10 ft. skin of dam	2000 psi. used for hearting of dam
Compression	1000 psi.	500 psi.
Shear	500 psi.	250 psi.
Tensional	80 psi.	40 psi.

Specific gravity of silt = 2.7

Angle of internal friction (f) = 33°

Ratio of earthquake acceleration to acceleration due to gravity, *i. e.* = a/g = 0.15.

Ultimate strength of concrete in shear = 400 lbs/ sq. in.

Coefficient of friction (C) = 0.65

Ultimate shear resistance = 400 lbs/sq. in.

Methods and results :

Trial of stress distribution, sliding factors and factors of safety against failure for both spillway and abutment sections were made for the following loading conditions:

Case A. Normal Pond (El. 1269) without earthquake and with uptail water at El. 1128.

Case B. Normal Pond (El. 1269) with earthquake, no earthquake and with tailwater at El. 1128.

Case C. Normal Pond (El. 1269) no tailwater, no earthquake and with silt (El. 1230) split.

Case D. Max. flood conditions (HWL El. 1230), no earthquake and tailwater at elevation 1167 feet. Due to hydraulic jump this is considered at elevation 1128 feet for stability calculations of the spillway sections.

Case E. Reservoir empty, with uplift, no headwater, no tailwater, no silt, and no earthquake.

All cases were also analysed with earthquake, this condition being given the suffix " 1."

Consideration of results.

Stability :—The lowest value of the factor of safety against overturning is 1.41, which occurs in the spillway section under condition D1. This is a condition including all known forces, some of transitory nature, so that the dam is considered safe against this form of failure.

The lowest value of the factor of safety against sliding also occurs under this condition D1 and is 5, which is above the figure of 4 used by the United States Department of the Interior, Bureau of Reclamation (referred to as USBR).

For the rock formation the cohesion is reduced to 200 psi. and the foundation of the spillway section is given a rise of 1 in 13 in the downstream direction, to resist the thrust, giving a safety factor against sliding of 4 for the head condition without earthquake which is accepted by the USBR.

The bulkhead foundation is given a rise of 1 in 20, which results in factors of between 4 and 5 with earthquake.

Stresses :

(i) *Allowable Stresses.* For the two grades of concrete to be used in construction, the allowable stresses are as given in Table A.

(ii) *Calculated maximum Stresses*—See Table B.

(iii) *Stresses due to differential Deflection.* The steep banks of this dam give differences in the depths at each end of a 40-foot block of the bulkhead section in the order of 35 feet. This results in a difference in the deflections at the ends of the block giving stresses due to the twisting action induced.

These stresses were investigated and the maximum found to be 63 psi. in shear.

(iv) *Stresses in Inspection and Access Galleries.* The inspection gallery in the main dam is located near the upstream face between elevations 1135 feet and 1148.5 feet, and is illustrated in sketch No. 1.

The stresses are obtained from the results given in the Engineering Monograph No. 12 published by the USBR.

Considering the two cases, D1, E1 :

Direct Stresses

	Case D1	*Case E1*
At apex.	215 psi –	111 psi
At centre of base...	127 psi –	112 psi
At mid side. ...	4 psi +	251 psi
At bottom corner...	298 psi +	372 psi

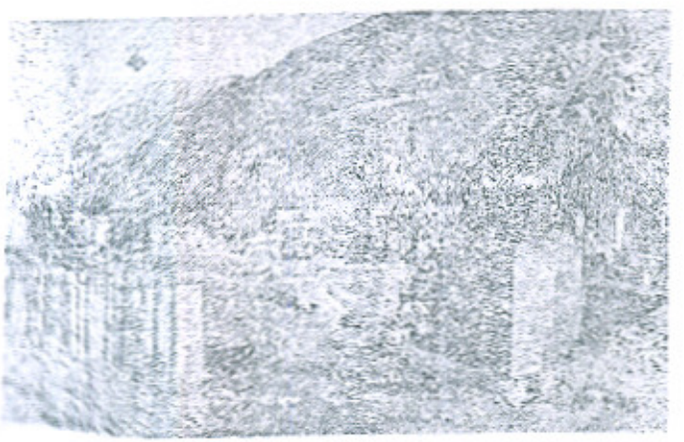
Compression shown—positive. Tension shown—negative.

These give a maximum tension of 8,680 pounds per foot run of gallery, and 3/4 inch bars at 12 inches are required at top and bottom with nominal steel at sides.

The access gallery is in an area which has a maximum principal stress of 100 psi parallel to the downstream face. By a similar analysis to that of the inspection gallery, the force per foot run is found to be 8,450 pounds, which also requires 3/4 inch bars at 12 inch centres.

(v) Temperature Stresses :—

A study of the temperature conditions of mass concrete of the dam indicated that with a placing temperature of 55 to 60 degrees Fahrenheit, a rise of 15 to 20 degrees Fahrenheit due to hydration, the drop to the mean annual air temperature of 72



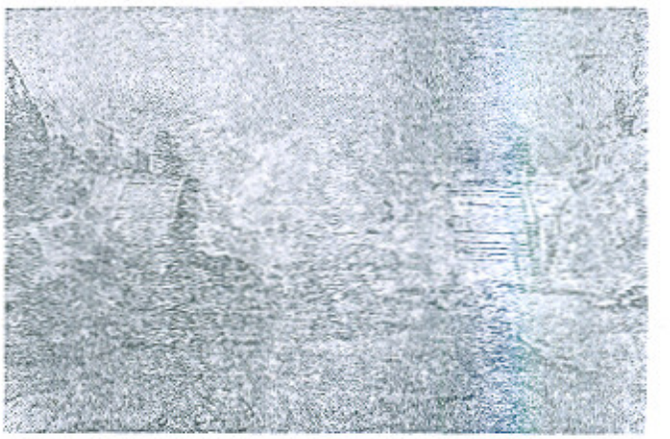
degrees Fahrenheit will be a maximum of 40 degrees Fahrenheit. This will result in a stress at the foundation equal to 100 psi tension which is permissible. This figure is sensitive to rises in the placing temperature and all precautions were taken to keep this down and reduce any rise in the concrete temperature after placing.

Conclusions:

The dam will be safe under all known possible conditions of loading as the following conditions have been maintained in the field.

(1) The spillway base does not extend below elevation 1050 feet and the bulkhead base below elevation 1132 feet.

(2) A good sound rock formation was established for the base, which was cleaned and all seams and fissures were plugged or grouted.



(3) The formation was given a rise to resist the thrust as calculated.

(4) All horizontal construction joints are sloped upwards in the downstream direction and treated in accordance with the concrete specifications.

(5) The concrete is of minimum strength, as specified.

(6) Adequate precautions have been taken to avoid tension due to temperature variation and shrinkage of the concrete.

Hydraulic Design.

Spillway : The spillway is located in the main river channel. This arrangement permits the use of the hydraulic jump pool at a favourable location and results in the safest and most economical device for dissipating the energy of the spillway discharge without destructive erosion. The channal of the spillway, confined between two concrete training walls, is on the downstream side of the spillway section of the dam proper and the hydraulic jump apron.

The structure consists of 9 bays 40' wide fitted with 40' wide and 40' high

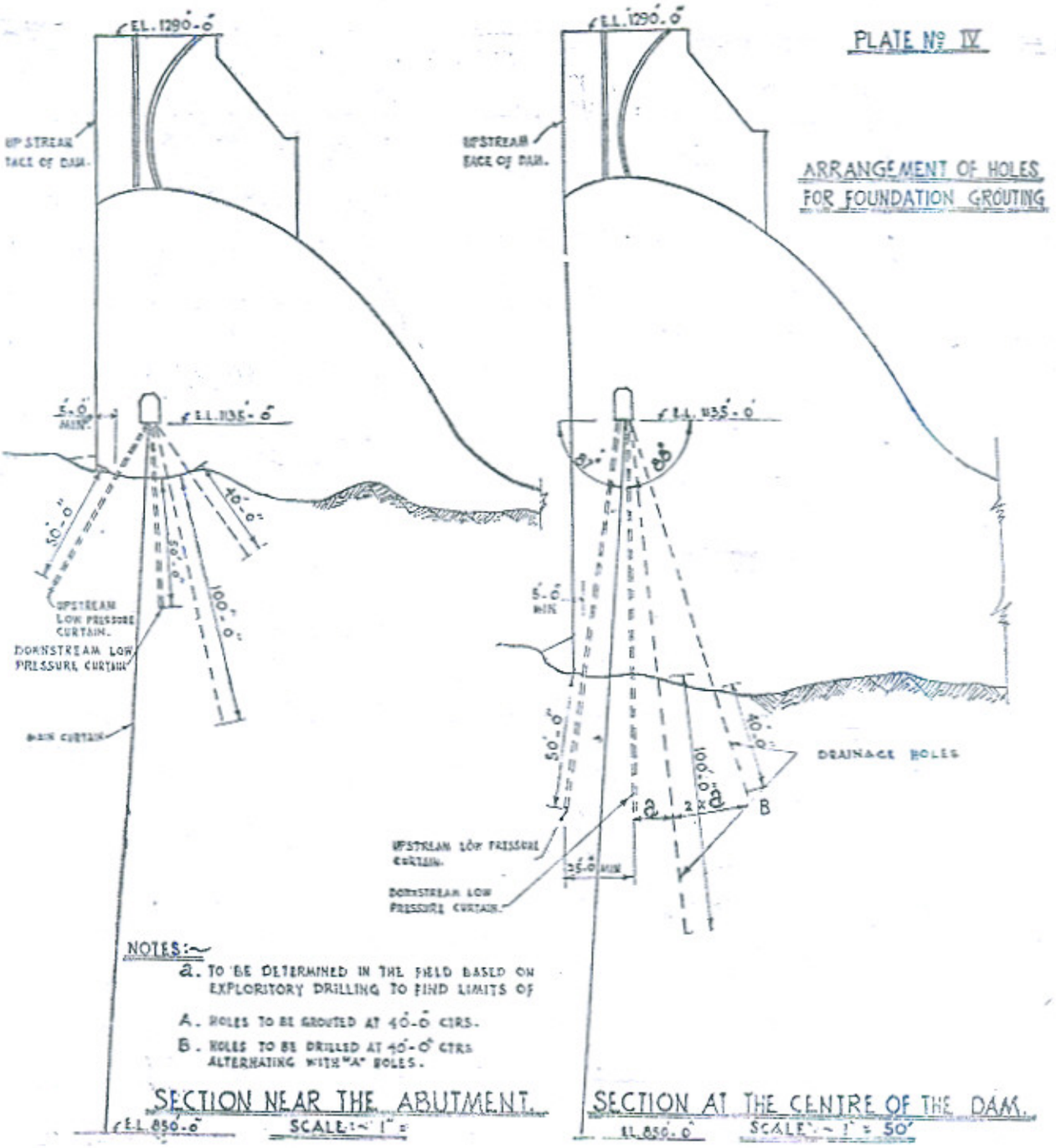
Tainter Gates separated by 10 feet wide R.C.C. piers. The maximum discharging capacity is 5,40,000 cusecs with gates fully open.

The upstream face of the spillway section is vertical, the crest is rounded with an ogee face downstream as shown in Fig. 2. The crest is formed to fit the shape that the overflowing water would take under conditions of maximum discharge if the dam were replaced by a weir with a sharp crest at the junction of u/s face of the dam with the rounded top surface.

The reverse curve joining the downstream face of the dam to the apron has a convenient radius of 18'-8", so that the pressure required to change the direction of flowing water is not so great as to give

PLATE No IV

ARRANGEMENT OF HOLES FOR FOUNDATION GROUTING



NOTES:-

a. TO BE DETERMINED IN THE FIELD BASED ON EXPLORATORY DRILLING TO FIND LIMITS OF

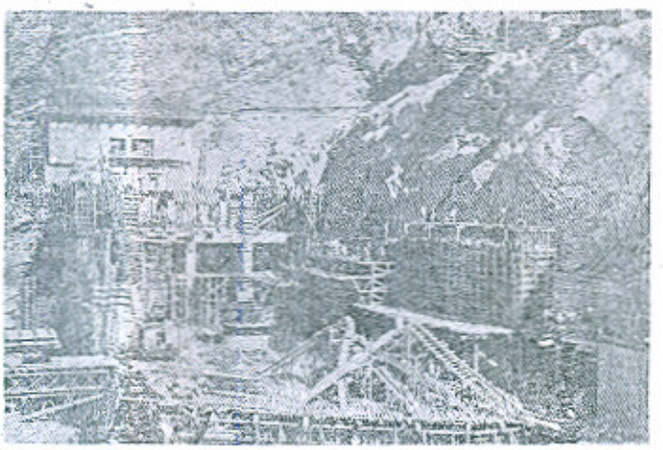
A. HOLES TO BE GROUTED AT 40'-0" CTRS.

B. HOLES TO BE DRILLED AT 40'-0" CTRS ALTERNATING WITH "A" HOLES.

undesirably high pressures in the joints at the downstream side of the dam.

Tainter Gates :

Nine nos, Tainter Gates will be installed as crest gates on the free overfall spillway section of the concrete gravity dam. They will be used to maintain reservoir level at elevation 1269 feet by regulating the spillway



discharge during periods of flow in excess of that required for power production or storage replenishment. The maximum flow which can be passed over the spillway when the reservoir level is at elevation 1269 feet is 320,000 cubic feet per second. During periods of extreme flood, the gates will be raised clear of the nappe to allow all out unrestricted discharge over the spillway. The maximum flood flow for which the spillway has been designed is 540,000 cubic feet per second. At this flow the lip of the gates must be raised to elevation 1280 feet to be clear of the nappe.

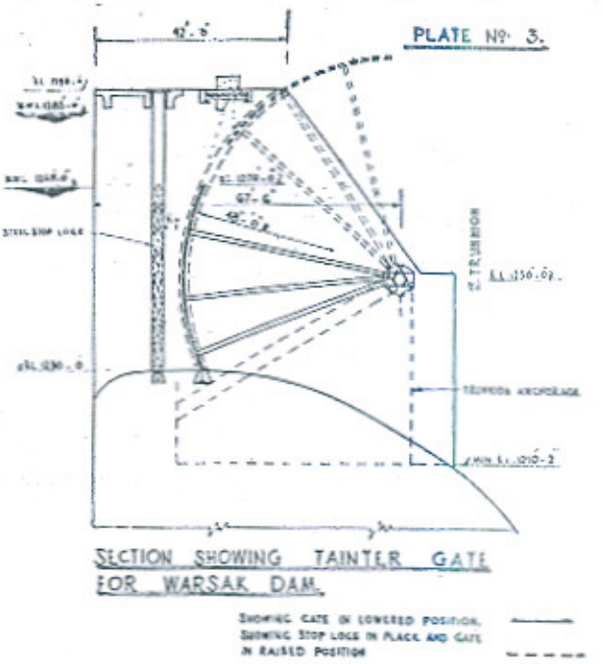
Sufficient steel stop-logs for one gate opening shall be provided to allow inspection and maintenance of the Tainter Gates.

The conventional form of Tainter gate consists of a skin plate formed of a segment of a cylinder and vertical element being circular arcs, supported by a framework of horizontal or vertical purlins and stiffeners. Purlins in turn may be supported by vertical or longitudnal girders over which two or more radial struts converge downstream of horizontal shafts or pins that are anchored in the piers and carry entire thrust of the water load. The skin plate is made concentric to this pin, and hence the resultant of the water pressure passes through the pin, creating no moment to be overcome in hoisting of gate. This is the fundamental principle of the gate. The hoisting load consists solely of the weight of the gate, the friction between the side seals and the piers, and the small force necessary to overcome the moment of the friction resistance on the pins.

Figure 3 shows the general arrangement of the gates, trunnion anchorages, hoists and stop-log slots.

Grouting :

The general plan for grouting the foundation rock under the dam provides low pressure grouting over the entire area followed by high pressure deep curtain grouting.



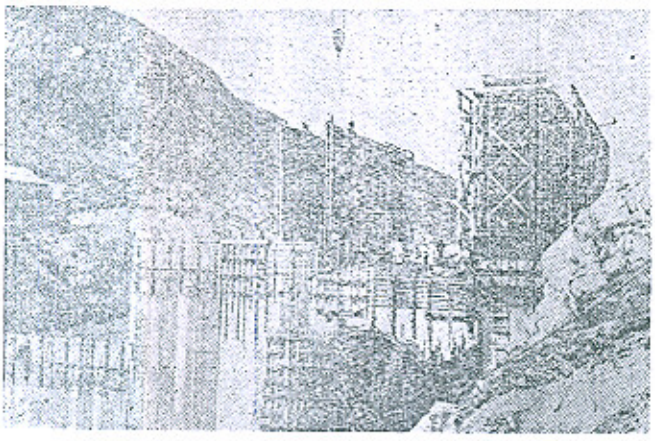
The design provides that prior to the placing of any concrete under the dam, the near surface rock of the foundation under the main dam be grouted by means of shallow holes upto 50 feet in depth below the rock surface. The purpose of this relatively shallow groutiong is to secure general consolidation of the upper layer of the rock so that peneteration and more effective sealing might be secured with higher pressures of grouting under the main

cut off curtain. The actual number and spacing of the holes, and the pressure to be used for grout injections, were determined by the nature of the shallow depth rock as revealed by excavation and results of water pressure or the tests of the grouting operation.

The main curtain or grout curtain was provided by the high pressure grouting of deep holes reaching a minimum elevation of 850 ft. at 20 ft. centres on a line 22' downstream and parallel to the axis of the dam. These holes with a downstream dip of 87 degrees from the horizontal were drilled from a gallery with floor elevation at 1135' to a depth of 224 feet under the maximum section increasing uniformaly to 285 at the end of the abutment sections.

Drainage.

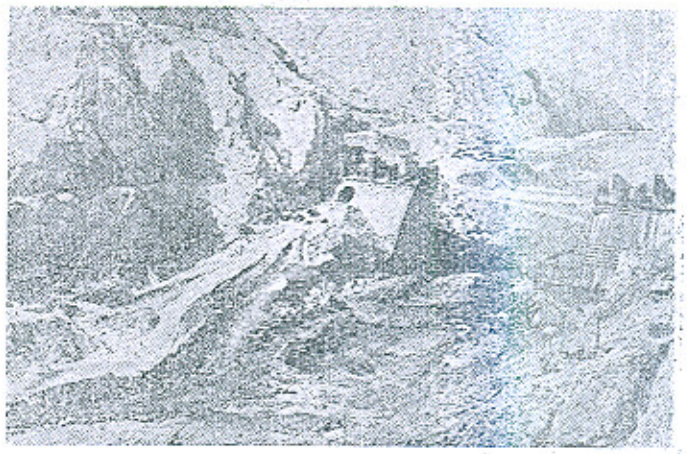
For drainage the design provides for two series of holes A & B (Fig. 4) at 40 feet centres opening in the inspection gallery on a line 22 feet downstream from and parallel to axis of the dam, inclined so as to drain the foundation on the downstream side of the grout curtains through 6" dia. pipe placed in alternate blocks. Drilling was done from the grouting and drainage gallery, and in general to a depth slightly less than the deep grout holes as shown.



The drainage system for the spillway apron and the training wall consist of 12' diameter pipe (concrete or steel). The lines of pipe are perpendicular to the axis of the dam and spaced at 20 feet centre midway between the contraction joints of the drain. These are connected to the drains in the stilling basin floor. The stilling basin is drained by a system of drains cast in the slab at every contraction joint upto a distance o 354' from the upstream face. The water is released through th.se joints.

Contraction Joints:

The concrete section of the dam is divided into 14 separate Monoliths (figure 1). The joints that divide the dam into these blocks are vertical and normal to the axis of the dam. They also serve to divide



the dam into convenient working units during construction. Temporary objective in providing the joints, however, is to allow for the normal shrinkage due to cooling as the heat of hydration of the cement was dissipated from the structure. In this manner the cracks which are formed in the monoloth due to cooling and subsequent shrinkage are controlled as to their location and shape. Two Longitudinal joints are provided in the structure at 50' centres from foundation rock to elevation 1130' for transverse shrinkage as no cooling water pipes were available. In the spillway apron section contraction joints are provided which divide the apron into blocks 50 feet long and 20 feet wide. No provisions are made for grouting the joints in the main dam and the apron. Every joint is made completely watertight by the insertion of plastic waterstops.

Acknowledgements.

The writer is indebted to Messrs H.G. Acres & Co Ltd, Consulting Engineers, of Canada and the Warsak Dam Project Organisation for their kind permission to draw material from their design reports and drawings. The valuable assistance rendered by the staff of design office of Warsak Dam Project Organisation is also acknowledged.

THE USE OF STABILIZED SOILS IN ROAD CONSTRUCTION

By
Mr. A. S. Khan

IN modern road construction, as in many other fields of human endeavour, the aim is to employ mechanized processes of ever higher efficiency. Only in this way is it possible to obtain the high rates of output with a minimum labour force that are essential if the cost of road construction is to maintain a consistent level in relation to the price of other commodities. The latter is dictated to a considerable extent by the cost of mass-produced goods produced by highly efficient mechanized methods, and, if the cost of road building is not to become disproportionately high, equally efficient methods must be employed.

Since 1947, mechanical methods of laying concrete and bituminous surfacings have taken the place of the hand methods formerly used, and the stage has now been reached where road foundations can also be completely constructed by mechanical means. The days of hand-pitched stone foundations are past, and even the use of a bulldozer to spread a layer of crushed rock is becoming outmoded by the latest mechanical methods, which are capable of laying foundation material to a highly compacted condition and with uniformly level surface.

The efficiency that can be reached by mechanical methods of road construction depends to a considerable extent on the suitability for mechanical handling of the materials used. From this point of view there are now only two types of base material which are entirely satisfactory for modern road construction; first, a graded mixture of crushed rock, either bituminous coated or wet uncoated, with a small maximum size (1½-2 inches), and secondly some form of "stabilized soil."

The term "soil stabilization" is generally taken to refer to any process where soil or other similar materials, which are inherently too weak to be satisfactory road foundation materials by themselves, are strengthened usually by treatment with an additive such as Portland cement, lime, bitumen or tar.

In this connotation, soil refers to any loose deposit in the subsoil that is suitable for treatment. Portland cement has been successcully used to stabilize a wide range of soils, including gravels, sands, silts and the light clays; hydrated lime appears to have a useful field of application in the improvement of the properties of heavy clay soils; bitumen and tar have both been successfully used to stabilize gravels and sands.

The other main group of materials suitable for stabilization comprises waste products such as quarry waste and colliery shale.

A number of reasons explain the increased usage of stabilized soil in road construction since the war. The first and most obvious is that in some areas there is a lack of good stone for building road foundations, and it would be too expensive to import the nearest suitable stone. This situation is quite common in certain areas of West Pakistan and there is no doubt that soil-stabilization processes will be used on an ever-increasing scale in these areas. The same situation exists in certain areas of England, particularly in East Anglia and in parts of south-east England, where an absence of hard rock necessitates the use being made either of the gravels and sand deposits or of the soils suitable for stabilization.

Another factor promoting increased use of stabilized soil for road construction in south-east England is the need to preserve the rapidly-diminishing sources of gravel and sand for making concrete for buildings and other constructional purposes where there is no alternative.

The third and most important factor favouring the use of stabilized soil is the low cost of construction compared with other methods of building road foundations. Road engineers employing the process for the first time almost invariably report a considerably reduced cost, sometimes as much as one-half. The main reason is, of course, that the soil which forms about 90 per cent of the stabilized materials is either free (when it exists on the site) or is very much cheaper than the materials used in traditional forms of construction. Apart form this, stabilization processes are highly mechanized, and rates of construction as high as 1,500 square yards per day have been achieved with a team of only six men, another factor which keeps down the cost.

SOIL STABILIZATION PROCESS

(*a*) ***Soil-cement.*** The most widely used material for stabilizing soils is Portland cement. Proportions by weight from 3 per cent for well-graded gravels to 15 per cent for clay soils are usually required to produce a material of adequate strength. The results are obtained in countries where the climate is comparatively wet; in arid areas, like the Middle East, a large amount of water has to be added in order to achieve adequate compaction of the mixed material and satisfactory hydration of the cement.

The presence of organic matter in soils sometimes interferes with the normal hydration process, but this difficulty can usually be overcome by the addition of a small proportion (2 per cent or less by weight of the soil) of calcium chloride.

Suitable proportions of portland cement, and of calcium chloride if required, are determined by making up mixtures of the soil with different proportions of the stabilizers and measuring the strength of the mixed material after appropriate compaction. The strength is determined by carrying out either a compression test on cylindrical specimens or a penetration test on specimens contained in cylindrical moulds, similar to the California bearing ratio test used for pavement design purposes. It is sometimes necessary to examine the durability of soil-cement, and test are then made to determine the loss in strength of the hardened material when subjected to soaking in water and to repeated freezing and thawing.

Methods of testing soil-cement, and all other forms of stabilized soil, have been standardized by the British Standards Institution; the tests are described in B.S. 1924 : 1947, and the British standard tests have been accepted as tentative international standards by the permanent International Association of Road Congresses.

The combined thickness of stabilized soil foundation and bituminous surfacing is usually determined by a method of flexible pavement design such as the california bearing ratio method. Such methods have been developed for designing road structures containing a stone base, and there is a fair amount of evidence to suggest that, where a soil-cement base is used, satisfactory results can be obtained with a somewhat reduced total thickness of construction as a result of the significant flexural strength of the stabilized material.

(*b*) ***Lime-stabilized soil.*** The addition of lime to a damp soil produces a hard material similar to soil-cement; the hardening process is, however slower than with cement. It is thought that the binding medium is produced as a result of a chemical reaction between the lime and the particles of clay. The advantages of the process are that (*i*) lime can often be produced locally in places in under-developed countries to which cement would otherwise have to be brought from long distances, and (*ii*) the addition of lime has a granulating effect on heavy, sticky clay which is difficult to deal with using any other stabilizer.

Fields in which lime stabilization has considerable promise are the provision of "working surface" and sub-bases for road constructed on heavy clay soils. Rain and construction traffic acting together on a foundation of heavy clay can transform a site into an impassable morass which retards work. The wet clay may eventually have

to be removed and replaced by granular material, at considerable expense. Lime treatment immediately after removal of the topsoil may reduce this hazard. The additional strength conferred on the soil by the lime may also be found to permit the thickness of stone macadam base to be reduced.

Reliable methods of evaluating stabilized soil using hydrated lime have not yet been developed fully in Great Britain or overseas ; but research is in progress, and, when the time factor has been taken fully into consideration, the methods evolved may well be found to be similar to those needed for cement-stabilized soils.

(*c*) *Bitumen-stabilized soil.* When the site of the proposed road is on sandy or silty soil it is possible to use between 4 and 10 per cent of some form of bituminous material to stabilize the soil. The additive is normally a cutback bitumen (*i e.* asphalt dissolved in a petroleum solvent such as paraffin) or bitumen emlsiufied in water. In some instances fluid tar has been used. Two types of processes can be recognized, the "wet" and "dry" aggregate methods, both of which involve the coating of the soil particles.

Under the climatic conditoins of north-west Europe soils are damp for the greater part of the year, and moisture adheres to the soil particles more strongly than does the bitumen when the latter is mixed with the soil. Methods of assisting the bitumen to adhere to the soil have been developed, one of which involves adding a small proportion of hydrated lime to the soil. In the Middle East, and in some tropical countries, however, the climate may be arid and the soil very dry. In such cases little difficulty is experienced in obtaining adhesion between soil and bitumen, provided suitable mixing machines are used.

(*b*) *Stabilization with chemicals.* Since 1945, a considerable volume of laboratory research and some field experiments have been undertaken with the object of developing organic chemicals capable of stabilizing soils. Trials are still in progress, the outcome of which is uncretain, but a major factor that will have to be taken into account in assessing the results of the treatments being developed is the cost of the chemicals. Cement, lime and bituminous materials cost in general between Rs. 100 and Rs. 400 per ton whereas chemicals produced by a series of complex reactions may cost between Rs. 1,500 and Rs. 8,000 per ton. Consequently, the only applications of these chemicals in road construction that can be foreseen are those in which they could be successfully used in concentrationsvery much smaller than those of other additives.

(*e*) *Application methods.* The materials used for the efficient construction of a road in any particular area have to be selected for economic reasons which include the cost of obtaining or manufacturing the material, the cost of transporting it to the site of the road, and the costs of processing at the site. Thus crushed stone is the preferred material in areas within a reasonable haulage distance of quarries, whereas the use of stabilized soil becomes advantageous in areas (for example those with mainly a young sedimentary geology) into which good stone may have to be hauled for long distances.

The choice of stabilize is conditioned partly by economics and partly by the type of soil involved and the prevailing climate.

Sandry soils are frequently stabilized with bituminous meterials, particularly in arid areas, where, in addition to acting as a binder, they assist in providing sufficient fluid lubrication of the sand grains to permit adequate compaction. Understandably, bituminous stabilization has been particularly applied in areas within reasonable haulage distance of oilfields and refineries, for example in the middle East.

The full potentialities of lime stabilization have not been realized, but apar. from the possibilities of dealing with clay soils in the United Kingdom there are reasons for believing that it will prove to be particularly applicable in the remoted areas of Africa and South America to which haulage of cement or bitumen is uneconomical but where a local lime-burning industry could be established.

Methods of Construction

There are two different methods by which stabilized soil roads are constructed. One is known as the mix-in-place method consisting of mixing the soil and the

stabilizing agent *in situ* with some form of rotary tilling machine; the other is known as the plant-mix method consisting of mixing soil from the site, or soil or waste material from a local source, with the stabilizing agent in a stationery mixing machine, and then spreading the mixed material on the soil formation. In both cases, the mixed material is compacted to a high density with a suitable form of roller or impact compactor.

Mix-in-place method. A considerable area of stabilized soil roads has been constructed very successfully using the agricultural type of rotary tiller for mix-in-place work. This machine is used first to break the soil to a fine tilth, and later to mix-in the stabilizing agent and water, if required. Each stage in the process requires a number of passes of the rotary tiller, and this type of construction is thus termed "multi-pass" to distinguish from "single-pass" construction, in which a specially designed plant carried out the entire process of mixing and compaction in one pass.

A compacted layer 6 to 8 inches thick can usually be produced by the mix-in-place method. For housing estate and other minor roads cement-stabilized soil of this thickness has proved adequate, even in madium clay soils, to carry satisfactorily both the heavy lorry traffic during the building of houses and the subsequent normal type of urban traffic.

Where mix-in-place constrution has been used for more heavily trafficked roads or for airfield runways the resulting stabilised-soil base of 6 to 8 inches has proved to be quite adequate. when provided with a two-course bituminous surfacing 4 inches thick, on the more stable granular subsoils. On weaker subsoils, where thicknesses of construction greater than 10 to 12 inches are required to sustain traffic loads, the mix-in-place stabilized soil forms what is known as the sub-base, and requires an additional layer of base material as well as the bituminous surfacing. In such cases the base may consist of granular material or plant-mixed stabilized soil.

A great advantage of mix-in-place method in a country with a wet climate is that the ultimate subgrade of the road is protected from rain and undue disturbance. It is, therefore, usually much stronger than it would be if exposed and damaged by construction traffic, as is common in traditional forms of road construction. Apart from this, mix-in-place construction is very cheap.

Plant-mix-method. The plant-mix method, as now normally used in Great Britian, comprises the mixing of the soil and stabilizing agent in a suitable mixing plant and the spreading of the mixed material with a bituminous finisher in layer 5 inches thick that compact to a thickness of 4 inches. In this way a very level surface is obtained on the finished base, and it is possible successfully to complete the road with a thin bituminous wearing course only 1½ to 2 inches thick.

Several types of stationery mixer are required, according to the type of soil being treated and the type of stabilizing agent being used. In the case of cement-treated soils, concrete mixers of the vertical drum type are suitable for fairly clean sandy gravel soils; pan-type mixers are more satisfactory for the more clayey gravels and for sands, and bath-type or continous asphalt mixers are most suitable for silty sands and light clays. Asphalt mixer are to be preferred for stabilizing soils with bitumens and tars. Quarry waste materials and clayey gravels, both of which can fall short of the requirements for stable base materials, are particularly suitable for stabilization by the plant-mix method.

The advantages of the plant-mix method are: (1) The ability to obtain intimate mixing of the soil and stabilizing agent, and (2) the even finish that can be obtained on the surface of the laid material. For these reasons, plant-mixed stabilized soil is most suited for use as a base; for main roads it is used in conjunction with some form of sub-base. In the latter case, it is not necessary to put down a layer more than 8 inches thick, since even with the most heavily-trafficked roads a stabilized-soil base of this thickness, together with a bituminous surfacing, is adequate to reduce the traffic stresses to a value that can be accepted by any recognized sub-base material.

EARTH MOVING MACHINES AND THEIR CAPACITIES

By

Mian Abdul Khaliq *

A description of earth moving machines and their capacities is particularly interesting and useful at this stage of our national development so that Engineers can form an idea about the correct choice of plant for the correct Job. In the last article, a description of the Bull dozer was given. The present article describes the use of the Excavator.

THE EXCAVATOR

An excavator is a machine primarily used for moving the earth, though the same machine can also be efficiently used as a lifting mechanism or for digging and loading the quarry material. On earth moving jobs, an excavator can be used as a dipper shovel, back-hoe, dragline or a clamshell, by the change of various attachments in the minimum amount of time. All these full revolving machines are available on Crawler mountings or rubber tire mountings. The important parts of an excavator, if it is to be used as a dragline, clamshell or a crane, are the Crawler mounting which include the Crawler frame, the Crawler Carbody and the Crawler treads or the rubber tire mounting, the lathice boom made up of steel angles, angle corners and tubuler braces, the fairlead to line up the drage cable, the bucket with drag chains ranging in capacity from $\frac{3}{8}$ cyd to 50 cyd, the sheaves, the engine fitted in a cab with gears and clutches, the rope drums, the drag drum, the hoist drum, the boom line, the hoist line, the dragline, the dump line, the A-frame etc., etc. In the case of a clamshell the reaving system is somewhat different from the dragline and instead of a bucket there is a clamshell which is worked with the boom drum, the hoist line drum, the digging line drum, the hoist line, the digging line, the tag line and the bucket closing socket cable. In the case of a Crane a hook for lifting the weight is attached with the hoist line. The study of the dipper shovel and a hoe is left for some other occasion and all the efforts are concentrated in explaining briefly the working of a dragline and the methods of calculating its output.

The dragline has the greatest digging range than any other attachment with which any other excavator can be worked and it can dig considerable distances above and below the machine line. It can work very efficiently under water also inspite of the difficulties such as inability to see the work, weaker penetration of the bucket due to decreased bucket weight and loss of material carried out of the bucket by water and sloppy condition of the spoil. The digging is done by pulling the bucket towards the machine and it is essential for better production to do the digging by layers instead of working in deep trenches. If the bucket does not fill in three lengths it is better to lift swing rather than waste time trying to get a slightly larger load.

The bucket should fill in a travel of 2 or 3 bucket length. Whether the bucket will remain in the carrying position or partially dump while being raised, depends on the opposing forces acting through the dump cable. The weight of the bucket front pulls down on the arch end of the cable and the tension between the hoist and drag cables tends to stretch its drag-yoke to dump sheave

*Executive Engineer, I.B.

section and pull the bucket up. When the bucket is full it should not be pulled a foot more as further dragging of the bucket will wear out bottom, waste power and raduce bank yardage. Several weights of buckets in each size are available for various classes of service and the heaviest type for the available machine should be carefully selected to get the maximum production. Before working with any type of bucket it should be ascertaimed that bucket teeth are sharp and built up to proper size. The boom angle for that particular bucket load should be adjusted before starting the operation. It is preferable to work with boom at highest admissible angle that will allow the reach and accuracy needed for a particular job. With the low boom (30° to 40°) we have longer reach, a better control of the throw of the bucket, alongwith more accurate grading and dumping. A high boom (40° to 50°) increases stability of the machine, decreases swing loads and swing power requirements. The dump cable and the hoist cable should also be of proper length for the efficient performance of the machine. Proper lubrication and maintenance will also help in increasing the production of a dragline.

The reaving system of this machine can be detailed briefly in the following words. The dump cable runs from the top of the bucket arch over the dump sheave and forward to the drag-yoke. The boom line is a standard four part rigging. The hoist line runs from the hoist drum over a large boom point sheave and down to the dump sheave case. The drag cable runs from the drag drum through the fair lead to the drag-yoke.

The dragline cycle includes lowering, digging, hoisting and dumping of the bucket.

Lowering. In any ordinary dragline cycle the bucket is not thrown or casted. Instead of this it is thrown very carefully and before lowering the bucket for digging it is ascertained that it is in true dump position. For this purpose the bucket is lifted with the hoist cable by keeping the drag cable slack. The bucket is then thrown to the ground under the boom as the best digging area is under the boom point. The bucket will rest on the ground on its teeth and arch. A pull on the drag cable will tip the bucket forward or backward into its teeth and the lip will dig in as it is dragged towards the machine. By keeping the hoist cable slack and by pulling the drag cable the bucket will continue cutting deep depending on the resistance of soil, sharpness of its teeth and the weight of the bucket itself. If so much cut is not needed then put some tension on the hoist cable which will raise the bucket slightly depending on the length of the dump cable. A long dump cable will raise the bucket from the rear by the hoist chains and a short cable will cause an upward pull on the arch and raise the front of the cable.

Digging. The bucket is lowered into the pit with both lines taut the hoist brake is wholly released, then re-applied smoothly as the bucket is about to strike the ground, and the drag brake is released enough to allow the bucket to drop straight instead of following an arc centering on the fair lead. When the bucket rests on the ground the hoist cable is slackened slightly and the drag clutch engaged. The drag cable pulls the bucket with the teeth digging in and cutting a layer of the soil which accumulates in the bucket.

Hoisting. When the bucket is filled, the hoist clutch is fully engaged and the bucket is lifted clear of the ground. If the bucket has a tendency to dump, the drag may be left engaged until the angle between the drag and hoist cable is sufficiently wide to hold it. The drag clutch is then released and the hoist is continued with the drag brake applied just enough to allow the hoist to pull the bucket forward and upward under the boom, without slackening the lines enough to dump it. The drag brake should not be very tight otherwise the bucket will strike the boom. The swing is started as soon as the bucket is clear of the ground. This is the period of heaviest load in the dragline cycle, as the machine is simultaneously lifting a load, pulling against the drag brake with the hoist clutch and swinging.

Dumping. Hoisting is discontinued as soon as the bucket is high enough to be dumped. When the swing is completed, the drag brake is released partially or wholly and the bucket swings out and dumps. For better operations the bucket should not be raised unnecessarily before dumping.

The approximate dragline cycle time in seconds, 90° swing and no delays is tabulated below :—

APPROXIMATE DRAGLINE DIGGING CYCLE
Time in Seconds

Size of dragline bucket.	Light Moist Clay or Loam				Sand or Gravel				Good Common Earth				Hard Tough Clay			
	45°	90°	135°	180°	45°	90°	135°	180°	45°	90°	135°	180°	45°	90°	135°	180°
	DEGREE OF SWING															
	Seconds															
⅜ yd.	16 Sec	19 Sec	22 Sec	25 Sec	17 Sec	20 Sec	24 Sec	27 Sec	20 Sec	24 Sec	28 Sec	31 Sec	— Sec	— Sec	— Sec	— Sec
½ yd.	16	19	22	25	17	20	24	27	20	24	28	31	—	—	—	—
¾ yd.	17	20	24	27	18	22	26	29	21	26	30	33	25	30	35	39
1 yd.	19	22	26	29	20	24	28	31	23	28	32	35	27	32	37	41
1¼ yd.	19	23	27	30	20	25	29	32	23	28	33	36	27	32	38	42
1½ yd.	21	25	29	32	22	27	31	34	25	30	25	38	29	34	40	44
1¾ yd.	22	26	30	33	23	28	32	35	26	31	36	39	30	35	41	45
2 yd.	23	27	31	35	24	29	33	37	27	32	37	41	31	37	42	47
2½ yd.	25	29	34	38	26	31	36	40	39	34	40	44	33	39	45	50

DRAGLINE OUTPUT

(Cubic yards per Hour) 90° swing

		⅜	½	¾	1	1¼	1½	1¾	2	2½
		5.0	5.5	6.0	6.6′	7.0′	7.4′	7.7′	8.0′	8.5′
Light Moist Clay or loam	...	70	95	130	160	195	220	245	265	305
		5.0′	5.5′	6.0′	6.6′	7.0′	7.4′	7.7′	8.0′	8.5′
Sand or gravel	...	65	90	125	155	185	210	235	255	295
		6.0′	6.7′	7.4′	8.0′	8.5′	9.0′	9.5′	9.9′	10.5′
Good common earth	...	55	75	105	135	165	190	210	230	265
		7.3′	8.0′	8.7′	9.3′	10.0′	10.7′	11.3′	11.8′	12.3′
Clay, hard, tough	...	35	55	90	110	135	160	180	195	230
		7.3′	8.0′	8.7′	9.3′	10.0′	10.7′	11.3′	11.8′	12.3′
Clay, wet, sticky	...	20	30	55	75	95	110	130	145	175

Note :—Bottom figures denote depth of cut. Top figures denote yards per hour.

Selection of the right Machine for the Right Jobs.

After a brief study of the dragline we now proceed to explain something about the slection of a machine. In order to get the maximum yardage an Engineer should try to use the maximum size of the bucket with the maximum boom length available by adjusting his machine at the correct and safe boom angle. For this purpose we should first know the dragline bucket lifting capacity. The weight of the bucket and load usually should not exceed 67% of the tipping load of the machine. Under favourable conditions, where there is solid footing and no suction on lifting of the bucket the dragline and clamshell load approach 75% of the tipping load as is the case with crane load. The tipping capacities of a machine can be studied from the attached diagram No. 1. On the basis of 75% tipping load of the machine a 2½ c.yd. excavator should not use a dragline bucket weighing more than 16000 lbs. with the distance of the hoist line from the centre line of the machine as 60 ft. If instead of 60 ft., operating radius is required as 70 ft. then the load of the bucket should not exceed 12800 lbs. Similar load can be found for a 2 c.yd., 1½ c. yd., 1 c. yd., ¾ c. yd., and ½ c. yd. machines. From the diagram No. 2 dragline zone we find that a machine having operating radius of 70 ft. and bucket load of 12800 lbs should not have more than 72 ft of boom length if the angle of boom is 24°. Maximum length to which a boom can be lifted is 69 when the boom length will be 100 ft, angle of boom as 39° and operating radius as 81 ft. Different ranges have been given for the clamshell and crane. The hourly dragline output for various solid is tabulated below :—

(Continued to page 26)

A Review on the Tube Wells for Drinking Water Supplies in Dacca, East Pakistan

By

Mr. Nazir Ahmad Jiabajee, B.Sc., P.S.E.-I

EAST Pakistan consists of the Delta of Brahmaputra Ganges, Manga River, Karatoya River, Surma River, Tista River, Padma and other rivers now extinict. Brahmaputra starts from lake Manserowar which is about 14900 feet above sea level and its area is about 133 square miles and maximum length 16 miles. The delta of Ganges and Brahmaputra covers an area of 50,000 square miles, composed of repeated alternations of clays, sands and mare with recurring layers of peat, lignite and some forest beds.

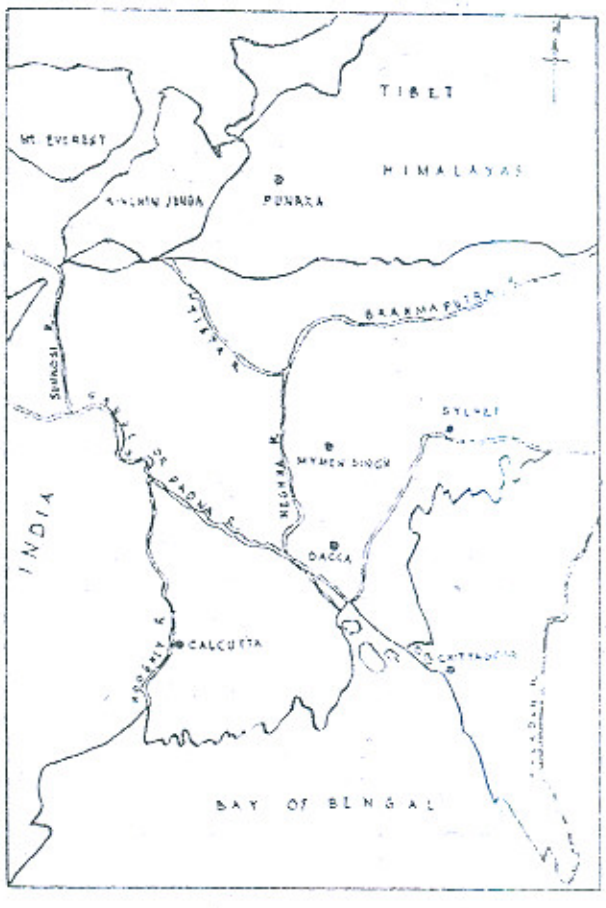
Southern Bengal has been reclaimed from sea at a latter date in the geological history by the rapid southern advance of Ganges and Brahmputra delta, through deposition of enormous load of silt. This has happend in about 5000 years.

An interesting investigation was done and the following details are taken from the figured section by Lieutenant Baird Smith regarding boring done in Fort William, Calcutta commenced in December, 1835 and completed in April, 1840 :—

In Dacca area winter-rain fall precipitates in December and is due to North East Monsoons and is about an inch. In June due to South West and South Monsoons give from 10" to 20 of rainfall. The annual rain fall increases as we follow the catchment area of Brahmaputra from 20" near Dacca Area to sbout 80" in the higher altitudes. As the ground has been made up of alluvium the water supply source is the ground water which permeates into the ground from rain or what is brought down by the natural drain such as Brahmaputra and its tributaries. Dacca area is almost water logged, the spring level appears at places, at 6 ft. only.

Activities of the Public Health Branch of Engineering are very recent and during the past few years some tube wells have been sunk, but as this is the first work of its kind in this area the borings have not got the full technical consideration of the latest type with the result that the discharge is meagre and full value of the investment is not being realised.

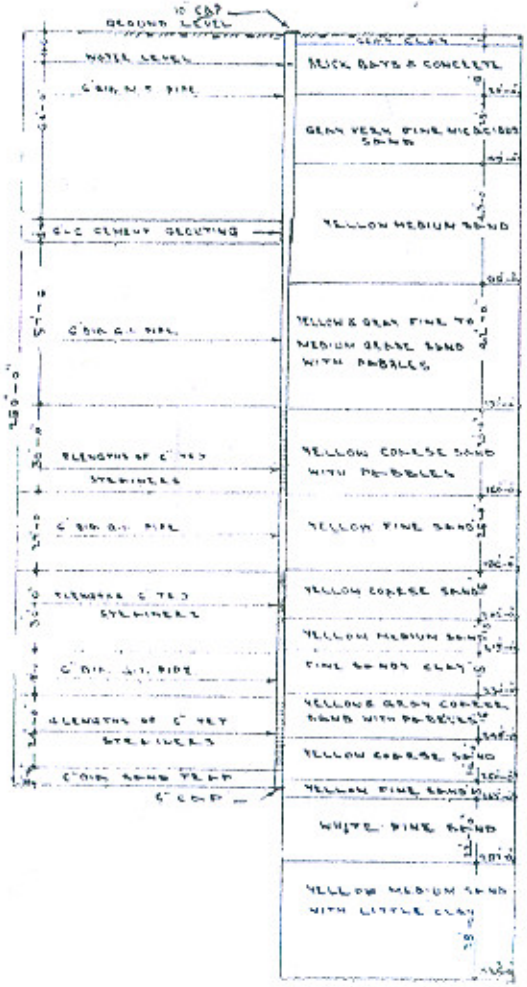
As records show alluvial deposit is very fine, and it may be silt, fine sand, medium sand, coarser sand marl and in some cases kankar or pebbles in lower reaches. They are mixed in various proportions and it is sometime difficult to extract water out of



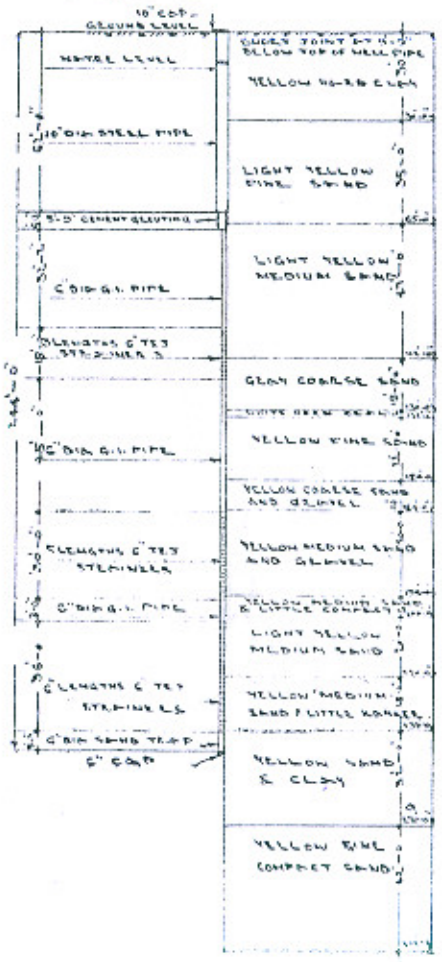
GEOLOGICAL SECTIONS OF TUBEWELL AT DACCA

SCALE — 40FT TO AN INCH

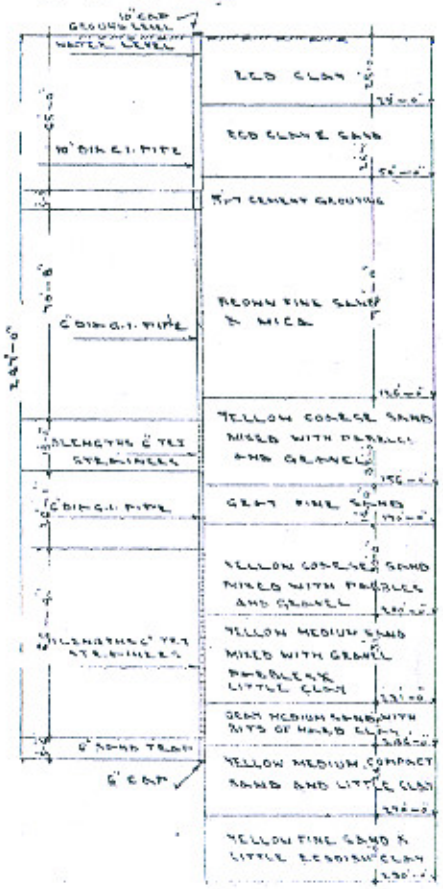
6" DIA TUBE WELL AT MITFORD HOSPITAL



6" DIA TUBE WELL AT KAKRAIL



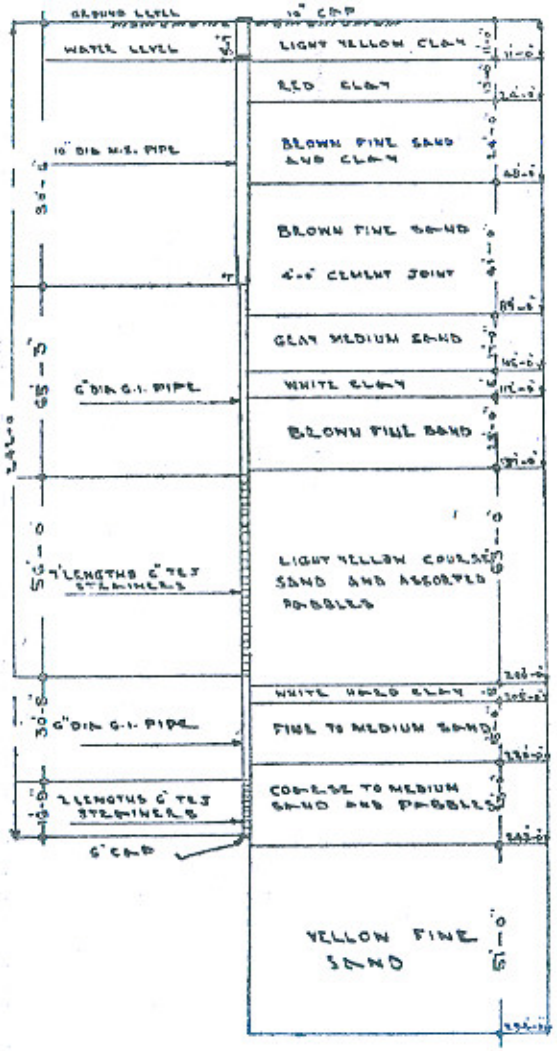
6" DIA TUBE WELL No 2 AT AZIMPORE



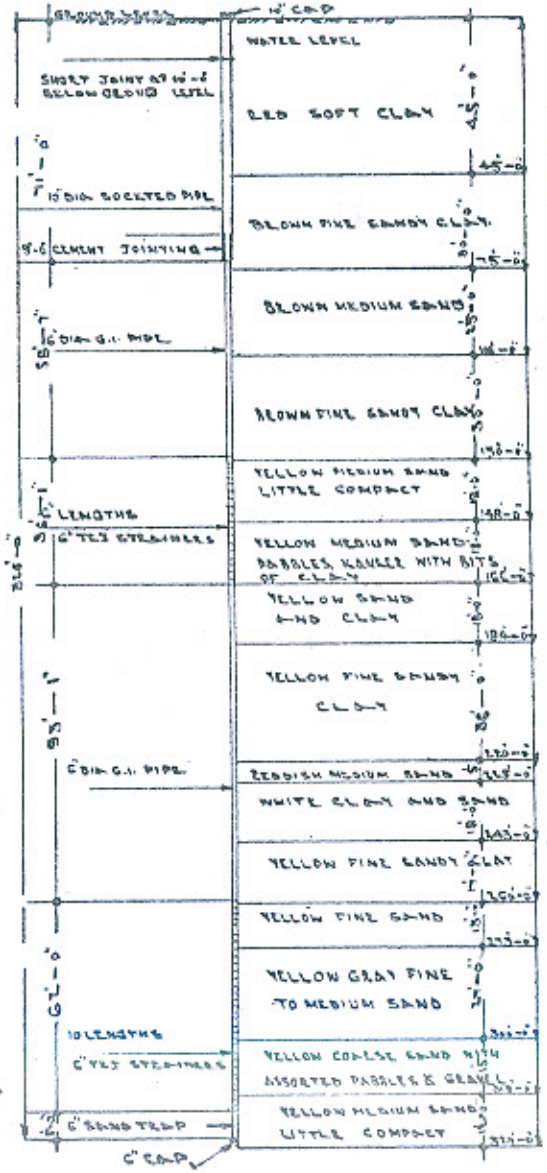
6" DIA TUBE WELL AT M.C. HOSPITAL



6" DIA TUBE WELL AT TEJGOAN INDUSTRIAL AREA



6" DIA TUBE WELL No. 1 AT AZIMPORE



COMPARATIVE STATEMENT OF STRAINER LOCATION

TUBEWELL LOCATION	STRAINER LOCATION	CLAY BAND	NATURE OF SOIL	SPELL LEVEL
MITFORD HOSPITAL	130-160 = 30 FT. 186-215 = 29 FT. 250-280 = 30 FT.	NO CLAY BAND	YELLOW COARSE SAND YELLOW COARSE & MEDIUM SAND YELLOW GRAY COARSE SAND	9'
KANKRAIL	100-118 = 18 FT. 163-193 = 30 FT. 201-237 = 36 FT.	ONE CLAY BAND 2'	LIGHT YELLOW MEDIUM SAND GREY COARSE SAND YELLOW MEDIUM SAND & GRAVEL LIGHT YELLOW MEDIUM SAND KANKAR	13'
TEJGOAN INDUSTRIAL AREA	137-193 = 56 FT. 227-243 = 16 FT.	2 CLAY BAND 6'-5'	LIGHT YELLOW COARSE SAND & PABBLES MEDIUM COARSE SAND AND PABBLES	9'
AZIMPORE No. 1	130-166 = 36 FT. 260-320 = 60 FT	NO CLAY BAND	LIGHT YELLOW COARSE SAND & PABBLES	7'
AZIMPORE No. 2	142-160 = 18 FT. 186-252 = 66 FT.	NO CLAY BAND	YELLOW COARSE SAND KANKUR & GRAVEL YELLOW MEDIUM SAND WITH GRAVEL & PABBLES	6'
M. C. HOSPITAL	89-101 = 16 FT. 118-134 = 16 FT. 213-237 = 24 FT. 320-352 = 32 FT.	YELLOW STICKY CLAY UP TO 53 BELOW GROUND LEVEL	LIGHT YELLOW MEDIUM SAND YELLOW & GREY MEDIUM SAND LIGHT YELLOW MEDIUM SAND LIGHT YELLOW COARSE SAND WITH PABBLES & GRAVEL	8'

the subterrarian main layer from the argyllicious mud or sticky clay. Deeper borings have not been attempted but it is certain that to a very large extent deeper boring for agriculture use also could be located in East Bengal with an advantage for drinking water supplies, for agricultural purposes specially to grow wheat crop during the season when rain is not available. Given below in Sheet I are the geological sections of various boring in Dacca which have been done in the last few years and these consist of the following tube wells which were executed by the Public Health Engineers of East Pakistan :—

6" Dia tube well at Kakrail, Dacca.

6" Dia tube well in industrial area Tejgaon, Dacca.

6" Dia tube well at Azimpur No. 1, Dacca.

6" Dia tube well at MC. Hospital. Dacca.

Enclosed sheet gives the profile section and a comparative statement of strainer location.

The following inferences can be drawn from an overall study of the above borings.

(1) The borings have been done which are rather of the small diameter The top casing is of a smaller size. The blind pipe is of a smaller diameter.

(2) Galvanised Iron Pipe have been used as blind pipe which are costly and serve no special purpose. Even wrought iron or black pipes could have been substituted to reduce the cost.

(3) No shrouding has been done around the different strainers length so as to protect the strainer from choking, or decay, and to arrest the flow of finer particles outside the strainer area.

(4) No compressed air developing has been done to wash out some small percentage of silt, fine sand and other harmful materials that can be extricated by compressed air development of the strainers.

(5) In some cases there has been no clay band which separates out the top water from the water at a lower stratum in the bore hole. The shallow subsoil

	Thickness in feet.	*Depth in feet.*
Surface soil, loose sand, and clay	10	...
Adhesive blue clay	15	25
Ditto with peat	10	35
Adhesive clay	5	40
Dark clay with decayed, wood intermixed	10	50
Calcareous clay with kankar	10	60
Green silicious clay Silicious clay with kankar. Variegated arnaceous clay. — Details of thickness not given, the lower portion represented as much thicker than the others.	60	120
Argillaceous marl	5	125
Loose sandstone	5	130
Argillaceous marl	20	150
Arenaceous clay with weathered mica slate and modules of hydrated oxide of iron	20	170
Calcareous clay	5	175
Coarse friable quartzose, conglomerate	10	185
Micaceous clay	20	505
Soft sandstone	5	210
Ferruginous sand intermixed, with clay	90	300
Fine loose sand with, minute fragments of felspar and granite ...	25	325
Sandstone slightly aggregated (first fossiliferous stratum 350 feet)	55	380
Shelly calcareous clay	5	385
Carbonaceous bed	10	395
Coarse conglomerate third (fossiliferous stratum 430 feet) ...	86	481

water on the top is liable to bacterial infection from house drainage or flood pollution.

(6) Where there have been no clay bands below 60 ft. from the ground level, no strainer should have been located above 110 ft., as this enables the bacteriological infection to travel in the subsoil water.

(7) The water level in this area is fairly high. The least being 6 feet below ground and the maximum 13 feet below ground. This is a dangerous state of affairs as water logging would permit the bacterial infection to travel to a considerable depth approximately which may be upto 100 ft.

(8) The maximum depth to which boring has been done to locate 6" blind pipe and strainer is 366 feet, which is not a deep boring considering the absence of impervious clay bands in certain cases.

In view of the extensive borings that have been done in West Pakistan where the alluvial soil is slightly coarser than it has been experienced in East Bengal side, the following points should have been considered essential to get the maximum advantage out of any such enterprise in tubewells :—

(1) Boring up to 120 feet be done with a bigger dia pipe say 15" casing.

(2) From 150 feet below ground, telliscopic boring should be done up to 450 feet preferably with 10" casing pipe. For the completion of the tubewell the strainer to be located should be 7" dia and the blind pipe should also be 7" internal dia. The usual standard test for strata samples should invariably be done and shown on the vertical section.

(3) The usual standard tests for water analysis should be carried out in the acquifires where it is proposed to locate a strainer. The result of such analysis of the water sample should be embossed on the soil profile.

(4) The suitability and palatability of the water samples should also be given on the soil profile.

(5) Invariably the strainers of the slotted type should be shrouded with coarse sand which is retained on 1/8" square mesh. The thickness of this shrouding should not be less than 1/4" all round the strainer. The shrouding should continuously be done as the casing is extracted. Even in that area where blind pipe exists coarser material should be shrouded as it will definitely be advantageous.

(6) In case of overlap of the joint it is preferable to have an overlap of not less than 10 ft. and possible a reducer of steel be provided from pipe to pipe.

(7) In case the discharge that is proposed to be taken which is about 22,000 gallons per hour, the length of the strainer should be about 120 feet. In case the discharge is to be 22, 000 gallons per hour, the top casing should not be less than 15" diameter.

(8) In the above cases it is desirable that the casing up to 115 or 120 feet below ground should be bored vertically and the pipes should be truely circular, otherwise the bowls of a vertical spindle bore hole pump may foul with the pipe casing as well as the shift of the bore hole pump.

(9) After the strainer is located it is desirable to pump air from a compressor with sutiable air pipe and education pipe with a suitable head on the casing pipe. Air at 50 lbs per square inch should sludge out the entire muddy water from the stratum of fine particles of silt etc., so that nothing finer than the coarse material around the strainer is allowed to exist. Later on the air pressure be increased and the education pipe be lowered to a greater depth to work in different acquifires where strainers have been located, till the entire area around the strainer has been washed out and cleared of finer grains of silt, sand and any other organic matter. The above clearing out be repeated till the water got out of the bore is absolutely clear.

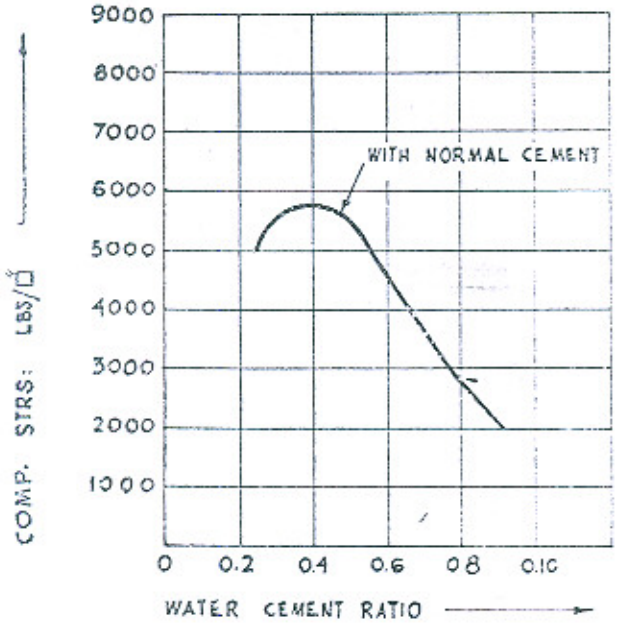
It is desirable to introduce the latest improved technique in tube well borings practised in West Pakistan, where in the Indus plain exhaustive experiments have been done in the last 44 years, in order to reap the fullest advantage for such work in the Indo-Gangetic Delta of East Pakistan.

THE CONTROL OF WATER CEMENT RATIO IN THE FIELD

By
Mr. Khalid Durrani,*

WATER-cement ratio as we all might know consists of the ratio (usually by weight) of the water to the cement for a particular batch.

Concrete consists basically of three components, the agregate, the cement and the water. Aggregate meaning the stones and the sand. Of all the components of concrete the cement and the water are probably the most important and upon these depend the strength of the concrete.



Hence we see that a relation between these two important components of concrete *i. e*, the water-cement ratio, owing to its affecting the strength is probably one of the most important aspects in the field control of concrete and has to be maintained.

A curve showing a relationship between the water-cement ratio and the compressive cube strength of concrete is given below.

The equation of the curve is either $U=\frac{A}{B^{x}}$ or $\frac{R}{X^{n}}$ where A, B, R and n are constant and x is the water-cement ratio.

It is found that the following factors affect the water-cement ratio of the concrete.

1. Free moisture in the Aggregates.
2. Grading of Aggregates.
3. Particle shape of Aggregates.
4. Consistency.

Moisture in the Aggregates.

Normally the aggegates obtained in the field have a certain amount of moisture present in them. This is owing to their being thoroughly washed before they can be brought into use for concrte manufacture. The stones or coarse aggregate do not have very much of free moisture present, because usually the water flows out, and moreover the surface area presented by them is rather small. This is not the case when the coarse aggregates are either finally washed on the final screen at the batching plants before finding their entry into the storage hoppers or when the aggregates are sprayed with water at the aggregate bins. In fact, to a certain limit, it is not the amount of free moisture in the aggregates, but it is the variations in moisture which brings about difficulties in way of field control of water cement ratio. Thus when concrete is being produced at a fair consistant consistancy with a reasonable constant amount of free moisture, it can well be imagined as to what will happen when the moisture in sand goes up from 5% to 9%, the result being that concrete will be too wet, so much so, that rejection of the same is desirable ; but when variations occur smoothly, the Inspector incharge can conveniently notice

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the gradual change in the consistancy of concrete taking place from batch to batch and is in a position to correct it without wasting the concrete or manufacturing concrete which may give inferior strength and other properties. Even minor variations in moisture content in aggregates may have serious effects on some mixes, thus 4" core concrete mix (used for the main dam core at Warsak and having 4" maximum size of aggregate) using 190 lbs. of water per cubic yard for 2" slump, would increase to 4" slump with a 1% increase in free moisture of sand alone and the water cement ratio will go up from the designed value of 0.63 to 0.67, and strengths will reduce by 200 p.s.i.

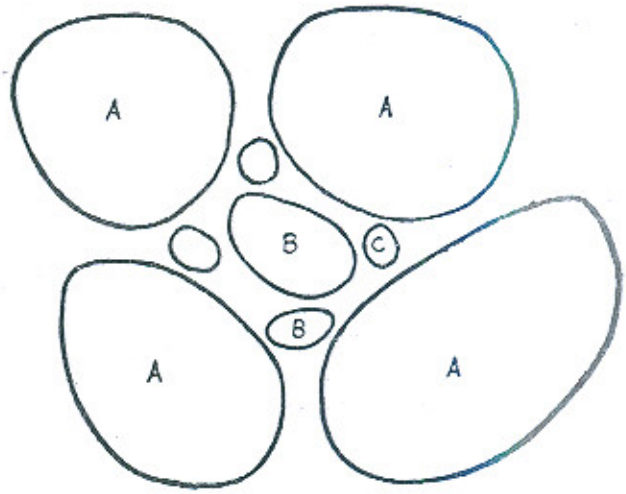
Large variations in free moisture of aggregates, especially sand brings with it yet another problem. For example the free moisture content of sand increses beyond 5%, then in addition to affecting the water content, quantity of sand will have to be increased, otherwise the sand content will fall below the optimum percentage. This entails placing difficulties, the concrete becoming too harsh and undersanded. Sometimes owing to large fluctuations of moisute contents in the aggregates, frequent changes in the batch scales of sand, water and stones is necessary. This interferes with the progress and output of the concrete and is least liked by the contractor.

Particle Shape.

The particle shape also plays an important part in controlling the W/c ratio. Thus concrete mix using rounded gravel and natural sand will require less water per cubic yard than the same concrete mix using angular rock and manufactured sand for the same workability. This phenomenon may be explained by the fact that rounded gravel has the least surface area and so is true of natural sand. Apart from this, natural rounded sand, in a concrete mix behaves like ball-bearings reducing the particle friction whereas crushed angular aggregate would need more sand and cement to achieve the same object. It is, therefore, quite clear that particle shape has to do something with the water requirements or in other words w/c ratio.

Grading of Aggregates.

Concrete, I would say, consists of a homogenous mixture of the stones, the sand, the cement and the water. The stones and the sand act as a binder, while the cement water paste which in grading is finer than both of them, acts as a cementing material. The strength of the stones is far more than the strength of the paste, therefore, to produce the strongest possible concrete, the stone size should be the largest practicable. Let A be the maximum size of the stones. Then the best possible way that these stones can be placed is as shown in the diagram. There should be space all around these stones in order to enable them to be cemented together. Again the best and the strongest possible way to fill the space in between the stones A is by having the maximum possible size that can fit in between them, let this size be B. The stone B should again have enough space all around to



be properly cemented together and so on. Thus we see that the aggregate should be such as to have a wide variation of size, without there being large concentration of any particular size. The cement water paste being the finest of the lot fills in all the voids thus cementing the mass together. If a smaller size of aggregate or an undue concentration of smaller sizes is used then the total surface area of the stones will be much larger and hence a larger amount of paste will be required. Again the larger the paste, the larger will the stone paste

ratio be, and hence the lesser the strength of concrete.

To control the strength of concrete the size of aggregate should be maximum and its grading uniform. It is only when we have these two characteristics then the surface area of the aggregate is minimum and hence the quantity of paste is again minimum. Thus for a fixed amount of cement, if we require a lesser amount of paste it would naturally mean lesser amount of water or a lesser w/c ratio.

Thus we see that in a uniform and in a non-uniform grading to bring about the same strength would require a lesser and a larger water cement ratio respectively, the max. size, the shape, the strength, etc., of the stones remaining the same for both the cases.

Consistency.

The grading and the moisture in the aggregates are in a way laboratory phenomenon and do not very much influence the placing of concrete. At the forms the property of concrete which is most predominant is consistency and upon this depends the facilitation of the conveying, the placing and the finishing operations. Consistency or workability is dependent on the amount of water added to the batch and is usually indicated by the slump. The larger the quantity of water the leaner the consistency, the lesser the water the stiffer the consistancy. Normally bearing in view the placing requirements, a definite consistency is adopted and continously maintained. It is at this consistency that we can maintain our designed water-cement ratio. If as is the tendency additional water is mixed to facilitate the working of concrete then naturally in addition to the water-cement ratio increasing there is also a possibility of the concrete segregating and settlement and bleeding may also occur.

Maintaining a definite amount of water for a batch does not necessarily mean that the consistency will remain constant. As previously mentioned any change in the aggregate moistures, the grading, the shape of aggregate particles, etc., changes the water requirements and hence the consistency.

Let us take an example of how a certain designed batch of concrete, having a definite strength and water-cement ratio, has to be modified in the field to obtain the same strength and the same water-cement ratio.

The concrete used for the Warsak Power House has the following designed requirements. Cement=1200 lbs. Sand=2100 lbs. Medium Stone=2400 lbs. Fine stone=1600 lbs. Water=650 lbs. Water-cement ratio=0.47. Slump=5"-6". Strength=300 p.s.i.

Medium stone is that material which passes a 2" sieve and is retained on a 3/4" sieve, while Fine-stone is that which passes 3/4" sieve and is retained on a No. 4 sieve. Sand being that which passes a No. 4 sieve and is retained on a No. 100 sieve.

Have we not seen that the aggregate we use have a uniform grading, varying in size right from 2" to about 1/100 th of an inch.

The aggregate obtained in the field had the following characteristics.

Medium stone—undersize and oversize-nil. Fine stone-oversize 45% undersize-nil. Sand-oversize=10% Free Moisture=5%.

In order to maintain the designed requirements in the ffield the following calculations have to be resorted to.

Sand :—

Adjustment of an oversize of 10%.

For every 90 lbs. of sand 100 lbs. have to be actually taken.

For every 2100 lbs. of sand $\frac{2100 \times 100}{90} =$ 2330 lbs. to be actually taken.

In other words we have 2100 lbs of S. S. D. sand and 230 lbs. of S. S. D. fines-stone mixed in it. Adjusting a free moisture of 5%.

The total mositure in the sand is $\frac{2330 \times 5}{100} =$ 116 lbs. Hence finally the scale weight of sand to be taken is 2446 lbs. of which 230 lbs. is F. S. and 116 lbs is water.

Fine Stone.

The designed weight of F. S. is 1600 lbs. but as found and above 230 lbs of F. S. is obtained from the sand.

∴ Net F. S. is 1600-230 = 1370 lbs.

This fine-stone has an oversize of 45%. *or* for every 55 lbs. of F. S. 100 lbs have to be actually taken.

or for every 1370 lbs. of F. S. $\frac{1370 \times 100}{55.}$

= 2490 lbs. ac tually taken.

Hence we have 1370 lbs. of F. S. and 1120 lbs. of M. S. mixed alongwith it.

The final scale weight of finestone thus is 2490 lbs. The final scale weight of M. S. being 2460—1120 = 1280 lbs. The designed water is 560 lbs. but it is seen that 116 lbs. of water are already present in the sand hence the scale weight of water is : 560—116 = 444 lbs.

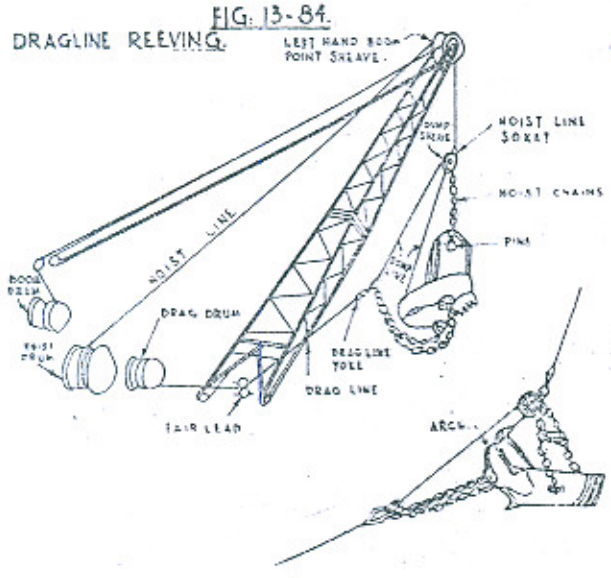
Final Batch weights hence are :—

Cement	=	1200 lbs.
Sand	=	2446 lbs.
F. S.	=	2490 lbs.
M. S.	=	1280 lbs.
Water	=	444 lbs.

From this example it can be clear how different field weights conform to the designed weights. And how it can be possible while maintaining the consistency, grading, particle shape, and moisture in the aggregates, to keep the water-cement ratio and strength of the concrete according to our designed values.

(Continued from page 16)

FIG. 13-84.



Rate for 1000 cft. of earth.

		Rs.
Delivered price of a 1055 L. C. dragline	=	550000
No. of working hours	=	10000
Interest, taxes, insurance	=	Nil.
Depreciation per hour at the rate of 20% per year	=	$\frac{550000}{10000}$
	=	Rs. 55
Depreciation per 8 hours shift	=	55 × 8
	=	440 (A)

Working cost of the machine including labour and supervision, replacement, fuel and lubrication = Rs. 25 per hour.

Per 8 hour shift = 8 × 25 = 200 (B)

Total A & B = 440 + 200 = Rs. 640

In case of good common earth output of a 2½ cyd bucket of dragline = 265 cyds = 265 × 27 = 7155 cft.

Per 8 hour shift = 7155 × 8 = 57240 say, 57000 cft.

Cost per 1000 cft. = $\frac{640}{57}$ Rs. 11.2, Say Rs. 11

Rate for 1000 cft. undressed earthwork average lead 200 ft. including lift and premium as per construction schedule of rates (I. B.) = Rs. 24-3-6.

Resale value of the machine has not been taken into account.

UNESCO IRAN SYMPOSIUM ON SALINITY IN ARID ZONES

By
Dr. Nazir Ahmad

UNESCO and Iranian Government recently organised a symposium on salinity problems of the arid zones. This symposium was the fifth of the series planned on an international basis to assimilate experts information on the subjects of immediate interest to Arid Zone countries. Previous to this symposium, UNESCO had organized four symposia at different times and places and the subjects discussed were :—

(*i*) Arid Zone Hydrology, held at Ankara in 1952.

(*ii*) Arid Zone Plant ecology, held at Montpeller in 1953.

(*iii*) Wind and solar energy, held at New Delhi in 1954.

(*iv*) Arid Zone Climatology and Microclimatology held in Canberra in 1956.

In addition to these, UNESCO has provided subventions for symposia organised by various scientific institutions in member states. About eight of these have been held in various countries. Last year one was held in Karachi on "Soil erosion and its control in Arid and semi-Arid Zones."

Arid zone research since its inception in 1948, is one of the major activities of Unesco. It actually came into active operation in 1950 after the appointment of a seven man Advisory Committee which has since been expanded to ten specialists on various aspects of arid zone research selected from member states. This Committee has been meeting off and on and at the time of salinity symposium, held its fourteenth session and chalked out the future programme.

During the symposium on salinity, the problem was discussed under four sub-heads, these being :

(*a*) Hydrology with reference to Salinity

Papers in this section were to deal with such problems as composition of underground and surface waters and origin of their salinity, artificial managements of the hydro logical cycle to prevent salinification, etc.

(*b*) Physiology of Plants and Animals in relation to Consumption of Saline Water

Papers in this section dealt with such problems as water relations and osmotic effects on plant growth on salt affected soils, mineral nutrition and specific on effects in plant growth on salt affected soils, tolerance of plants to saline and sodaic conditions, plant selection and cropping practices in relation to irrigation water quality, physiology of animals in relation to uptake of salt from water or plants, etc.

(*c*) Use Of Brackish Water In Irrigation With special reference to Saline Soils

Problems such as characteristics of water suitable for irrigation, influence of soil properties on the suitability of brackish waters for irrigation, irrigation practices, land drainage and leaching, soil management and amendment in relation to irrigation water quality, effects of brackish irrigation waters on soils, maintenance of permanent cultivation with brackish irrigation waters, etc., were discussed.

(*d*) Demineralization Of Saline Water

Latest developments in demineralization of saline and brackish waters by such pro cesses as vaporization with conventional

fuels, solar distilation, freezing, electrodialysis, chemical methods, etc., were put forth.

Iranian Government had invited twenty specialists from the regions of North Africa, the Middle East and South Asia. UNESCO arranged for specialists from Australia, France, Germany, Italy, Nether-land, U. S. S. R., U. K., U. S. A. etc. Members of the advisory committee also took part in the proceedings and in all seventy six persons attended the symposium. Twenty three persons attended from Iran proper. Thirty persons attended on the invitation of Unesco and other world organizations such as Unesco Science Co-operation centres at New Delhi, Cairo & Paris, F. A. O. (Rome), International Hydrology Organization, World Metrology Organization etc.

In all, there were fifty-six contributions on the four subjects mentioned before. There were fourteen papers submitted on the subject of hydrology with reference to salinity, eleven papers on the subject of physiology of plants and animals in relation to consumption of saline water, sixteen papers on the use of brackish water in irrigation with special reference to saline soils and eleven papers on the various processes of demineralization of saline water.

A list of papers with the subject matter and the names of authors is given at the end.

Summarising each contribution will increase the volume of this note considerably. However, below are mentioned some of the main inferences, adoption of which may be of help for our country. The UNESCO is going to arrange for the publication of these papers and it will be a useful world contribution giving an idea of the present state of knowledge about salinity problem.

IMPORTANT CONCLUSIONS WORTH ADOPTING IN PAKISTAN

A study of the hydrology of a country is essential to have better insight into the salinity problem. A convenient suggestion is to select certain small areas and study their hydrological aspect in preference to the whole zone which has several complications usually difficult to eliminate. Under certain hydrological cycles, it is possible to get fresh water from areas which possessed saline water in the earlier stages.

The contribution by Prof. Schoeller of France on the interpretation and chemical analysis of saline water needs careful study.

As regards hydrology and salinity of West Pakistan, it was stated that lack of sub soil drainage is a major cause for quick rise of watertable and the physical properties of soil grains such as freezing of water film, capillarity and low permeability have all helped in the concentration of salts of irrigation waters due to lack of drainage. Insufficient water supplies to the fields have contributed to accumulation during the last 50 to 75 years of the irrigation practice of the country.

It was stressed that the most successful drainage for this country is that by shallow cheap type of tubewells and it was suggested that all Persian Wells may be provided with baked clay or Munj String Strainers and worked by machinery. About 10,000 of persian wells in a doab are ready-made tube wells needing only a strainer worth rupees one hundred and a pumping set costing only Rs. 3500/-.

Use of baked clay strainers for extensive net work for direct irrigation, will set the stagnant sub soil water in motion, dewater quickly greater soil zones for redistribution of surface accumulated salts and circulation of water will start producing healtheir crops.

The next subject about the adaptation of plants to saline water was discussed by eleven contributors & needs carefull study for adopting practice of the foreign countries. The work of Prof. Henckel of Russia is particularly worth following as he illustrated how he has succeeded in producing healthier and better crops from specially treated seeds resistant to sulphates and chlorides. The contribution of Prof. Walter, Elgabaly, Snmonneau, Djezirei, Golegolake and Wahab have all advocated intensive study to produce crops resistant to the saline conditions of arid zones.

Under the third section pertaining to reclamation of saline soils and the use of brackish water for irrigation, sixteen papers were contributed, mainly by Russia, America, Francne, Italy, India, Pakistan, Austria and

Iraq. These contributions are most valuable to persons engaged on similar problems in this country and it will be worth summarising the suggestions of various authors as applicable to this country. The work is rather extensive and certain tables and figures have not yet been reproduced along with the papers. The author of this note is in correspondence with experts of foreign countries to get their complete papers after which it is proposed to review all the sixteen contributions.

The last item discussed in some details was about the demineralisation of saline water for irrigation. Eleven papers on the subject were contributed by experts of different countries. The modern method used such as Electrodialysis, solar distillation, freezing, ion exchange, membrane processes, osminic, Reverse Osmosis, use of nuclear energy and several other devices used in various advanced countries were discussed. Our country being still in a backward state cannot make use of all the processes suggested in the symposium. Use of solar distiller seems promising in a country like ours where sun shines all the year round and the temperature is very appropriate to develop this method. Further development of the device can result in the use of heat energy to pump water of the waterlogged area, some investigations on the subject are in progress in the Irrigation Research Institute also.

List of authors and papers read and discused during the UNESCO Iran Symposium at Tehran.

SECTION I

		No. of papers
1.	F. DIXEY "Hydrology with reference to salinity" ...	3
2.	L. J. TISON "Hydrologie et salinite des eaux souterraines" ...	12
3.	G. V. BOGOMOLOV "Conditions of formation of sweet waters under pressure in certain desert zones of North Africa, the U. S. S. R and South-East Asia" ...	21
4.	A. I. SILINE-BEKTCHOURINE "Conditions of formation of saline waters in arid zones" (communicated paper) ...	30
5.	H. HUMBER "The influence of the geological conditions on the salinity of the Zain Rud" ...	48
6.	T. N. JEWITT "A comparison of sub-soil water conditions and salinity in the Sudan and Iraq"	10
7.	H. SCHOELLER "L' interpretation des analyses chimiques des eaux salees" ...	6
8.	D. J. BURDON AND S. MAZLOUM "Salinity types of groundwater from Syria" ...	1
9.	J. TAVANA "An examination of geochemical data of Qum Area surface waters" ...	47
10.	J. MARGAT "Les eaux salees au Maroc" (Presente par R. AMBROGGI) ...	50
11.	R. G. DOWNES "Soil salinity in non-irrigated a rable and pastoral land as the result of unbalance of hydrologic cycle" ...	28
12.	J. K. JAIN "Hydrologic aspects of the salt problem in agriculture and its control through improved water management in arid and semi-arid regions (with particular reference to India)" ...	29
13.	NAZIR AHMAD "Soil salinity in West Pakistan and means to deal with it" ...	27
14.	A. LOEHNBERG "The reduction of salinization by modifying the hydrological cycle" ...	24

SECTION II

		No. of papers
1.	H. WALTER "The adoptation of plants to saline soils" ...	2
2.	P. SIMONNEAU "Essai sur la vegetation halophile" ...	19
3.	M. H. DJEZIREI "Les peuplements de paletuviers des cotes sud de I' Iran" ...	42
4.	H. GOLEGOLABE & A. ZARGARI "Les plantes des environs de Massileh et le changement de leur structure interne" ...	46
5.	P. A. HENCKEL & B. P. STROGONCV "Physiology of plants consuming saline water" ...	22
6.	G. REPP "Basic principles on the salt tolerance of plants" ...	4
7.	M. M. ELGABALY "Studies on salt tolerance and specification effects on plants" ...	41
8.	A. K. KHUDAIRI "Boron toxicity and plant growth" ...	13
9.	R. D. ASANA "Physiological problems concerning crop production under saline conditions in India" ...	25
10.	ABDUL WAHAB "Salt tolerance of various varieties of agricultural crops at their germination stage" ...	38
11.	D. A. DENTON "Adoption of ruminant animals to variation of salt intake" ...	31

SECTION III

		No. of papers
1.	V. A. KOVDA "Principles of the theory and practice of reclamation and utilization of saline soils in the arid zone" ...	20
2.	C. A. BOWER "Prediction of the effects of irrigation waters on soils" ...	36
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TRANSMISSION LINES AND OVERHEAD CONDUCTORS

By
Mr. M. H. Masud Butt*

THE function of a Transmission System is to provide facilities for conveying electric energy from the generating stations to areas in which power is consumed. Only forty years ago there was hardly any transmission line at a voltage higher than 132 kV. Today extra high voltage transmission lines exist in three continents at 220 kV, 345 KV, 380 kV and 400 kV. However, it is worthwhile to point out that older transmission system could not provide any particular method of design. Some of the most modern methods of design are being developed due to "the result of a race to keep step with ever-increasing demand of System Capacity". The modern development of inter-linking various power stations situated at distant places is called GRID. This method of operation provides minimum transmission losses, a reliable and stable service providing an economy in transmission of electric power.

Hydel, Thermal and Diesel power Stations are always connected in parallel in order to draw more power during the summer from hydro electric stations and in the days of low water supply, the power may be drawn from the fuel burning plants. Thus a large quantity of fuel can be saved. In this method of inter connecting various power stations, a minimum reserve generating capacity and standby power stations are maintained. In the future Grid of west Pakistan, Warsak, Dargai, Malakand, Rasul, Mangla and some other small Hydel projects, *viz*, Shadiwal, Kurrum Garhi; Chichokimalian, Gujranwalla, Taunsa, etc. will be interlinked with the giant natural gas burning thermal station of Multan along with thermal and diesel power houses of Lyallpur, Montgomery, Shahdara, Burewala, etc. This Extra High Tension Grid of West Pakistan envisages a network of transmission lines on Single Circuit 132 kV, Double Circuit 132 kV and 220 KV transmission towers. It will thus stabilise a reliable service on economical footings. Studies of network analyser can reveal the exact position of this grid.

In order to design a transmission line, the following steps are taken :—

(*i*) For a proposed load, investigations are carried out.

(*ii*) Route or routes of a line for the proposed load centres are made from the survey maps.

(*iii*) Size of conductor is determined from the magnitude and nature of load while keeping in view the transmission distance.

(*iv*) Estimates of cost must be prepared from a reasonable and reliable information of the current market price.

(*v*) Cost per mile of the transmission line must be determined.

(*vi*) The transmission line should be so planned that a safety factor in design must exist as it plays a vital role in a reliable but sound design.

(*vii*) The design must be economical and must be financially justified.

Besides keeping in view all these factors, a designer should make use of every other facility which is available to him. He should also pay a special attention to the structures before he should carry out a proper investigation of design.

The following points further derive his special attention :—

( ) Every structure used on transmission should be robust to withstand stresses imposed on its various parts by the line.

(*b*) Both structure and conductor must be of sufficient strength with a predominont factor of safety to withstand the loads due to wind, ice or by the line itself.

(*c*) Height of the structures should be sufficient so that adequate clearance

*Extra Asst. Director, CEA.

from the ground at maximum load or maximum temperature is available.

CONDUCTOR MATERIALS

Copper, aluminium, steel and other metals have been used on transmission lines but aluminium and steel both participated an important role in extra high tension transmission lines. Aluminium Cable Steel (A.C.S.R.) has almost a monoply in the field. Various types of conductors have been designed to withstand 55 KV, 132 KV, 220 KV, 345 KV, 380 KV and 400 KV. The cross-sectional area in each case varies thus changing a lot of specifications of these conductors which can be seen from any good catalogue supplied by the manufacturers, However, it is very difficult to compare various metals because electrical and mechanical properties are also taken into consideration. The question of cost also plays a vital role in this selection. British Standards Institution has published Standard Specifications for Copper, Aluminium, Steel-cord Aluminium and other types of conductors. Why these materials have been selected for transmission lines have been discussed in brief in order to give an idea about these metals :—

I. COPPER

Due to high electrical conductivity, elasticity, and great tensile strength, it has been given prime importance.

(*i*) It is worthwhile to point out that this electrical conductivity depends upon the purity of metal, as 0.01% of Phosphorous or 0.03% of Arsenic in copper can reduce the conductivity to 15%. Metallic impurities also further lower the conductivity to a great extent. In order to obtain a high quality of product, most of copper should be refined.

(*ii*) The ultimate tensile strength of hard-drawn copper is great per sq. in. of section in the smaller wires. The approximate breaking stress in lbs. per square inch is as follows :—

Dia. in Inches	*Breaking Strees in lbs. per sq. inch.*
0.258	58,000
0.204	60,000
0.162	62,000
0.128	64,000
0.102	65,000
0.081	66,000
0.064	67,000
0.051	67,500

(*iii*) The elastic limit of hard-drawn copper wire is about 60% of the breaking stress. However, it may be 70 to 75% of the ultimate stress.

(*iv*) The temperature co-efficient of copper depends upon the conductivity of copper. In case of hard-drawn copper the conductivity of the conductor improves to 2% to 4% during manufacturing process. The tensile strength is also improved. However, a conductor of 20 ton quality has a conductivity of 98% while that of 30 ton quality has 97%. At 20°C, the conductivity of hard drawn copper conductor is about 96% while its temperature co-efficient of resistance is 0.00393. According to "Dellinger" when,

n = fractional conductivity of a sample as compared to annealed copper in percentage,

L_t = temperature co-efficient for pure copper,

$n\,L_t$ = % conductivity $\times$ L_t

$= 0.96 \times 0.00393$

$= 0.00377$

At a temperature of 20°C, the temperature co-efficient for copper of 100% conductivity can be calculated by the formula given below :—

$$L_t = \frac{1}{234.5 + t}$$

$$= \frac{1}{234.5 + 20} = 0.00393$$

II. COPPER CLAD STEEL

It is manufactured by welding a coating of copper on a steel of wires. The line conductor made of this material are stranded. Experience and tests have shown that it is an excellent material for many purposes in over-head transmission lines. The following are the main characteristics for its selection :-

(1) These conductors have a great tensile strength than the equivalent pure copper conductors.

(2) Due to special copper-iron alloy, the conductivity is 30% to 40% of the pure copper conductor of the same diameter.

(3) Due to high tensile strength, it is very useful for river crossings and places where exceptionally long spans are required.

ALUMINIUM

This metal, in these days, is playing a vital role in almost every industry due

to its low price, light weight and many other mechanical properties. However, the following are the main reasons for its selections :—

(*i*) It is a cheap metal and thus reduces the cost of a transmission line.

(*ii*) Cold rolling and drawing slightly decreases the conductivity of aluminium but improves the tensile properties in the same way as in the case of copper at 20°C. This conductivity of wire is about 60% while its temperature co-efficient of resistance is 0.039.

(*iii*) At a particular efficiency of transmission line, the cross-sectional area of aluminium conductor will be larger than copper when conductivity of aluminium is small. The diameter of aluminium conductivity will be 1.26 times greater than the diameter of a copper conductor of equal resistance provided conductivity of aluminium is 60% while that of copper is 97%.

(*iv*) Though the sectional area of the aluminium is greater than copper, yet the density of this metal is too low. The weight of aluminium conductor will thus be one half the weight of the equivalent copper conductor.

(*v*) The tensile strength of aluminium is much lower than that of copper but the large sectional area of metal neutralises this difference to some extent. The ultimate strength of aluminium conductor is 75% of equivalent copper conductor.

Considering all these factors, the following drawbacks still exist in the use of this metal :-

(*i*) The increase in cross-section of conductor exposes greater surface to wind pressure. Due to this fact the supporting structures must be designed for a greater transverse load strength.

(*ii*) In the case of aluminium conductors, height of the towers must be increased for the same span of length when compared with copper conductor. As the linear co-efficient of expansion of aluminium is 1.4 times that of copper, so large sag in conductor will occur.

(*iii*) Additional cost is incurred in towers and the saving made in conductor thus counter-balances the total cost of the scheme.

iv. ALUMINIUM CABLE STEEL REINFORCED

The difference between ACSR Conductors and the pure aluminium conductors is that these are manufactured with the central wires made of galvanised high tensile steel while none in the case of pure aluminum conductors. ACRS is widely used in transmission lihes in all the countries where rapid pace of electrical development in power transmission is taking place. It consists of galvanised steel wires with one or more layers of aluminium wires outside. It has been developed to permit :—

(*a*) considerably cheaper conductors than copper conductors of equal resistance.

(*b*) the sag in an overhead transmission line is smaller because it is 5% stronger and 20% lighter than copper. Under the conditions of 3 8″ iceload plus 8 lbs. per sq. ft. wind pressure at 22°F when factor of safety is 2,an ACSR conductor of 0.15 sq. in. section for a 600 ft. span will have a sag of just over 7 ft. while that of copper will be 14 ft.

(*c*) Longer spans with ordinary height of towers are used thus reducing the total number of supports by 25%.

(*d*) 25% reduction in the number of supports will thus reduce the total cost of insulators, foundations, erection and maintenance cost, etc. to the same limit.

(*e*) Breaking load is considerably larger than that of the equivalent copper conductor. A steel-cored aluminium conductor of breaking strength 15,000 lbs. provides a greater margin against abnormal load, than a copper conductor of breaking strength 10,000 lbs.

(*f*) The weight of conductor is 25% less than the equivalent copper conductor.

(*g*) A point of vital importance in high tension lines for ACSR is that it has a higher corona limit. Thus in long distance transmission lines, it has the advantage of raising the critical voltage limits of the line. This increase may vary from 30 to 20 per cent.

(*h*) In order to calculate resistance and reactance of the conductor, DWIGHT has shown that the resistance may be taken as equal to that of the core and the outer wires connected in parallel while the reactance is equal to that of a non-magnetic conductor of the same outside diameter.

(*i*) As the diameter is comparatively larger for a given capacity, so it can be used for high voltages and high elevations.

In high tension lines "Dog" conductor has been used for 66 kv lines while "Wolf" and "Tiger conductors have been used for 132 kv lines. For the future grid of West Pakistan "Lion", "Goat", and "Lynx" conductors will be utilised for 132 kv and 200 kv transmission lines. Here is a brief summary of these conductors which have been named for a particular area of cross section nominated for particular electrical pressure.

Name of the Conductor	*Area of Cross-Section in sq. inches*	*Voltages on which U ed*
1. DOG	0.10	66 kv
2. TIGER	0.125	132 kv
3. WOLF	0.150	(-do-)
4. LYNX	0.175	(-do-)
5. LION	0.231	(-do-)
6. GOAT	0.300	220 kx.

V. Iron and Steel

Galvanised iron and steel conductors have been used for

(*i*) Extremely long spans due to high tensile strength.

(*ii*) Short line sections exposed to abnormally high stresses due to climatic conditions.

(*iii*) 11 kv lines in order to supply power to rural areas.

(*iv*) Short distances where copper conductor is smaller than S. W. G. No. 8 and is unsuitable due to its mechanical strength, it is most advantageous and economical to tranmit electric energy.

Besides all these facts, the following demerits are quite evident in such type of conductors :—

(1) Increase in resistance will occur due to current distortion which is very difficult to estimate.

(2) The skin effects cannot be calculated at all as the permeability of the material is unknown.

(3) Due to hysteresis losses, the resistance is further increased in A. C. and in the case of stranded conductors, by spirality effect. Thus potential drop shoud also be considered.

(4) The internal inductance will increase to a great extent though in non-magnetic conductors, it is negligible at normal voltages. Often the value of internal inductance exceeds the external inductance.

Due to all these factors, calculations and deductions should be based on test data obtained for different sizes and grades of conductors. For ordinary galvanised steel strand cable, it has been calculated according to Matthiessen's scale the conductivity is 11.5 per cent when the breaking strength is 70,000 lbs. per sq. in. It is used in guys for overhead lines. In the first World War, it has been experienced in Germany that it has not only unsuitable electrical properties but also costly and difficult to erect. The galvanising can easily be damaged and joints can be made with difficulty.

Size of Conductors

From a standard catalogue of any manufacturer the usual sizes of copper and aluminium conductors can be seen in which resistance and weights are given which are required for design calculations of overhead transmission lines. The following table can help a designer to some extent as a ready reference :—

Resistance and weight of conductors at 68° F (20°C)
Standard Conductors

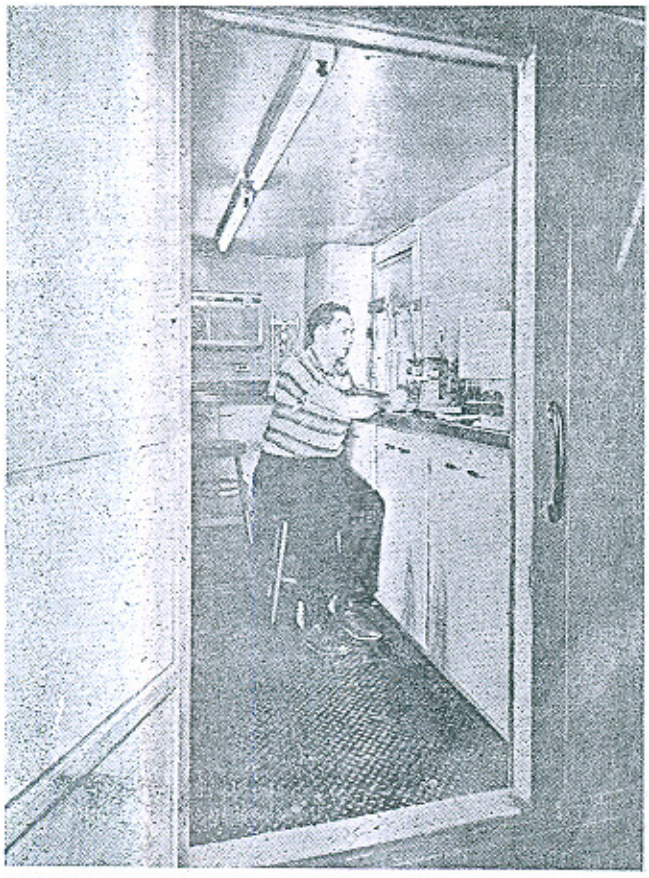
Size circular miles and B. & S. gage numbers	Dia-meter inch	Area of cross-section		Copper		Aluminium	
		Circular miles	Square inches	Ohms per mile	Weight per 1000 ft. pounds	Ohms per mile	Weight per 1000 ft. pounds
1,000,000	1.150	1,000,000	0.7854	0.0575	3.060	0.093	940
950,000	1.125	950,000	0.7461	0.0605	2.910	0.098	893
900,000	1.100	900,000	0.7068	0.0638	2.750	0.103	846
850,000	1.000	850,000	0.6676	0.0676	2.600	0.109	800
800,000	1.035	800,000	0.6283	0.0719	1.450	0.116	752

ENGINEERING TESTING IN A MOBILE LABORATORY

By
Thomas J. McNeil*

ALMOST everybody has been able to measure in terms of his own work the increased necessity for engineering testing in connection with programs of road building, bridge and dam construction, and airfield and irrigation projects. The post war years have seen more cognizance taken of testing and quality control by civil engineers than had the previous 25 years period. As a greater appreciation of the need for soil foundation work and quality control of building materials grew, so did the availability of testing equipment and trained personnel.

Today, there is a new concept in engineering testing—the concept of greater mobility, closer control, and faster results. Testing, in effect, must move to the job site to squeeze the greatest economy and the best control from engineering projects. To be effective in the analysis of compaction, for example, the results of a day's work should be known before the equipment leaves the site. To effect greatest control and the most rapid progress in concrete quality control, cylinders should be tested as close to the job as possible, with a minimum of disturbance due to transportation to central testing laboratories. Qualitative analyses of asphalt mixes are most valuable when obtained as soon as possible after the asphalt is delivered to the job site. In a work, quality control is most valuable and most economical when determined at the location of the job.



Performing soil test in Mobile Laboratory

This concept of mobility in lab. testing is not a recent development. Since before the war, a variety of efforts have been made to effect mobility in testing.

One of the earlier mentions made of a portable lab is made in *Soil Cement News* for September, 1943, where a portable field lab for control of operations on an airfield built in van Horn, Texas is described.

The British, for an example, report in the magazine *Roads and Road Construction* for June 1, 1945, the description of a mobile laboratory for testing concrete which was released by the Road Research Lab of the Department of Scientific and Industrial Research, in Middlesex.

In April, 1947, the Director of Public Works of Columbia, Georgia, J.M. Graddy, announced through the pages of the *American City Magazine* the use of a portable soil testing lab for inspection control of the soil cement street improvement program. Tests included soil sieve analysis, moisture content, density determinations at optimum moisture content, and other tests.

*Vice-President Soil test, Inc., Chicago.

C. D Reed, mechanical engineer and lecturer in civil engineering at the University of Liverpool reports in *Civil Engineering* and *Public Works Review* for May, 1953, on the mobile civil engineering lab for research projects and student demonstrations.

The State Highway Commission of Wisconsin, in the December 1, 1955 issue of the *Western Builder*, reports on the enthusiasm of contractors, engineers, and county and state highway officials in the saving of time effected by the use of a lab on wheels for testing bituminous mixtures used on the Wisconsin highway improvement projects.

The Corps of Engineers at Ft. Belvoir, Virginia has on prominent display a complete mobile laboratory in a 20 ton trailer body.

The Iowa State Highway Commission first built mobile labs in 1947 when they purchased three house trailers and equipped them for field work.

Most recently, the Washington Asphalt Company of Seattle reported that their mobile lab has resulted in the saving of thousands of dollars in rejected materials. It raised the standing of a contractor, Walter W. Castner, in the estimation of state officials, because he demonstrated, by running his own tests in this lab that he knows what he is doing and wants to do the best job possible. It also gave the contractor's operating personnel a new familiarity with the terms and significance of specifications and a new consciousness of product and quality.

In a word, mobility in testing is not really new, but slow aborning.

As manufacturers, we have been upplying mobile labs primarily to the foreign market for the past eight years. There are two reasons for this greater market abroad. One, there is a definite lack of commercial testing labs overseas. Two, there is an urgent need for engineering testing in connection with the economic development programs being encouraged by the U.S. in the under-developed nations. Since many American engineers are consultants to foreign governments, their knowledge of the need of engineering testing creates a greater demand for testing labs than would normally be the case. The range of work necessitated mobility, and hence the portable field lab has been emphasized.

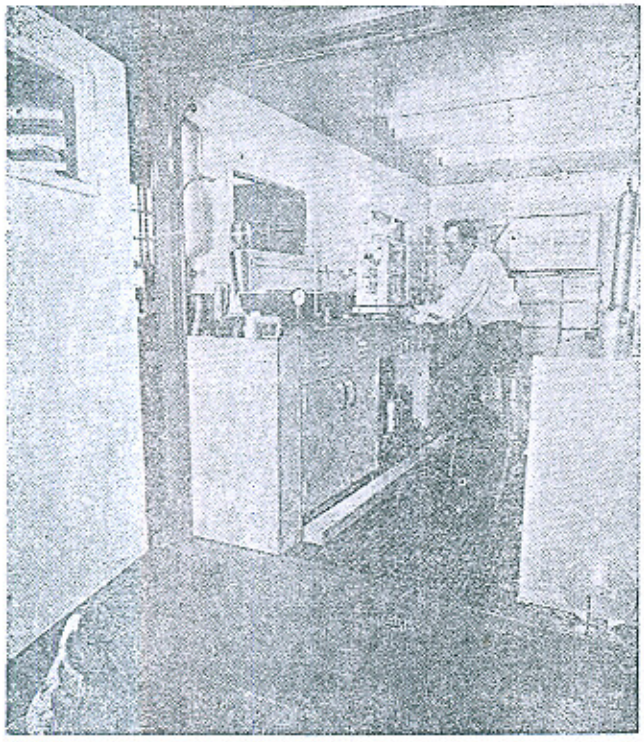
However, today in the U.S, several factors have combined to give greater impetus to the acceptance and use of mobile labs here. First, there is a growing recognition of the value of, and need for, on-the-job testing and quality control. Next, the workload facing all construction agencies, because of the Federal highway program verges on being staggering : some 39,000 miles of new highways have now been authorized by the Congress and every mile must be subjected to a multiplicity of precise tests, both in foundation work and in materials quality control. The standards, therefore, that must be met by country engineers who will participate in the secondary road program along with state highway departments are more rigid than ever before. Likewise, contractors will be subjected to more rigid control simply because of the magnitude of the program and the care that must be exercised against both over and under design.



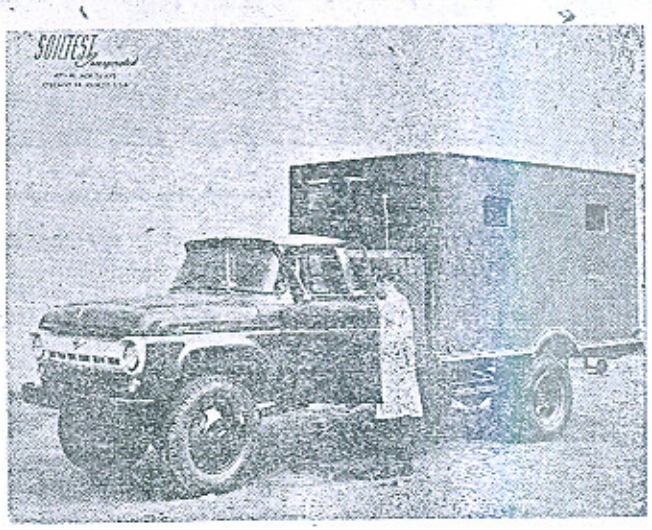
Inside view of mobile trailer laboratory

So, the use of mobile labs cannot but increase with this new program and greater appreciation of engineering testing.

At present, there is a mobile laboratory touring the U. S on exhibit at state highway departments, universities, and at certain conventions. This is a trailer lab which is felt to be more functional in this country where good roads are fairly well distributed and where it is not considered too economical to tie up a truck mounted lab when it will sit for weeks and months in the same spot. The trailer under discussion is 30 feet long, and completely fitted out with a utility system that includes a power generator, running water system, air compressor, air conditioner, space heater, and ice box. It is equipped with general lab apparatus including a dynamic sieve shaker, scales and balances, a lab oven, and a general range of glassware, thermometers, mixing bowls and various lab tools.



Weighing simples in Mobile Laboratory



Mobile Laboratory with All wheel Drive Truck

It also has an assortment of testing equipment for performing soil, concrete, and asphalt tests. The equipment that is fitted in a lab is dependent on the use for which it is especially intended. This depends on the engineer and the job.

The acceptance of this concept of a mobile testing lab was forcefully demonstrated during this tour of the United States where more than 800 engineers, state and county officials, contractors, architects, students, and teachers personally came to the lab to inspect its facilities. Time and again it was a visitor to the lab who pointed out the need that existed and the use what could be made of this testing concept. It underscored our belief, that mobile labs can be a valuable tool toward better engineering for the dollars spent.

BOOK REVIEWS

TABLES FOR THE DESIGN OF BEAMS AND SLAB By THE LOAD FACTOR AND ELASTIC METHODS according to British standard code 114 : 1957 by Jacques S. Cohen published by Concrete Publication Limited, 14 Dartmouth Street, London, S.W.I.

The load factor method of design can be applied as easily as the elastic method by the use of suitable tables. The tables given here have been prepared so that either the load factor method or the elastic theorey may be used as required. Examples are given to show the use of the tables and the advantages and economies that are possible by using the new code. This book illustrates the use of tables only and is not concerned with the theorey of load factor method of design.

EDITORIAL NOTES FROM CONCRETE AND CONSTRUCTIONAL ENGINEERING, by H. L CHILDE 96 pages, published by concrete Publications Ltd.— Price 7 sh. 6 d.

The main purpose in writing these articles has been to encourage engineers and scientists to explore beyond technical problems. The author's comments on the sayings and writings of technologists, some of whom are eminent in their professions, indicate the pitfalls that beset those who have not developed an inquiring and logical mind, and whose loose thinking results in statements that those who made them no doubt thought were profound truths but which are shown to be half truths or nonsense even when they appear in British standards and codes of practice.

These examples, and the constructive advice throughout the book, are intended to encourage and help young technologists to write and say exactly what they mean, and particularly to pursue course of study that will fit them for the higher posts that await technical men with more general knowledge and wisdom than their fellows.

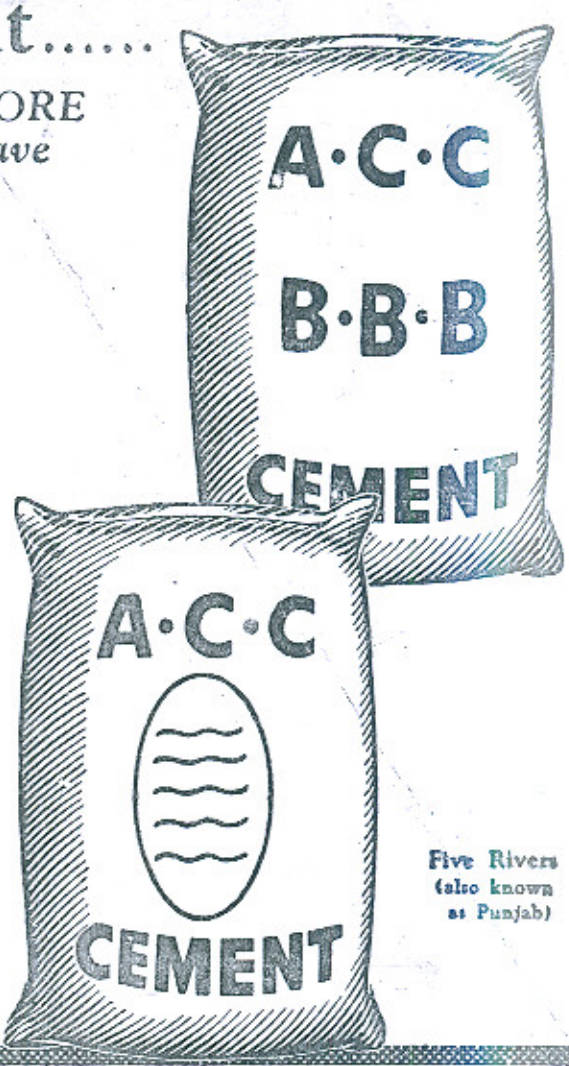
GRANDED EXAMPLES IN MATHEMATICS for National Certificate Students by E. G. SHALDERS Published by Blackies & Sons Limited, Glasgow.

A very useful set in three parts, these graded examples in Mathematics are meant for the use of National certificate students. The problems are so arranged as to enable the student to acquire a sound knowledge in the short time available. The exercises are arranged to facilitate a simple effective method of teaching, viz. a short period of instruction followed by a set of simple exercises, this process being repeated several times during the lecture period.

Presenting a syllabus in this way ensures that (*a*) sufficient mathematics is taught in time to be used by the student in the Engineering subjects, (*b*) each topic is dealt with twice, thus tending to keep up a form of continuous revision.

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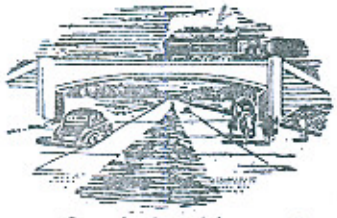
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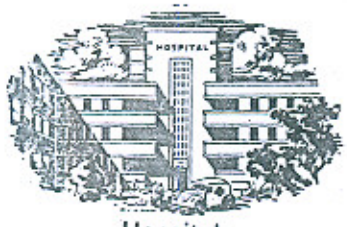
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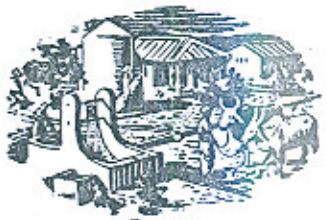


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