Paper No 316

Foundation Investigations
and
Salient Features of the design of Chichoki Power House Raft

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1. Brief Description

Chichoki Power House is located on Upper Chenab Canal Main Line Lower about 25 miles from Lahore between the Upper Chenab Canal crossings of G.T. road and Lahore—Sheikhupura road. The Power House will have an installed capacity of 12900 k.w. consisting of 3 turbo-alternator sets of 4300 k.w. each. The contract for the manufacture and supply of the generating and switching equipment has been awarded to Messrs. Rade Koncar of Zagreb, Yugoslavia.

The Upper Chenab Canal takes off from Marala Headworks and irrigates large areas of Rachna Doab. Its Main Line Lower is mainly a feeder channel for Lower Bari Doab Canal. There are several falls situated on its Main Line Lower which have been combined to give a single drop of 22 to 27 feet which will be utilized for generation of power.

The power house will be located on a loop channel off-taking at R.D. 2,18,000. This would not only facilitate the construction of the Power House and its subsidiary works but also add to its maximum head by allowing lining of the power channel and reduction of the slope to 1 in 10,000. The tail race outfalls at R.D. 2,28,000. Upper Chenab Canal Main Line Lower, from where the original channel will be deepend up to R.D. 251,000 by means of dredging. (Plate No. 1).

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Brief data of the Powerstation is as follows.

Discharge : Max/Min	...	8145 cs./3200 cs.
Head	...	22′ to 27′
Installed capacity.	...	12900 k.w.
Firm Power.	...	8000 k w.
Total utilizable annual output.	...	97.5 million k.w.h.
Total cost.	...	132 lacs (of Rupees)
Cost per k.w. of installed capacity.	...	Rs. 1100
Cost of firm power,	...	3.44 pies.
Cost benefit ratio	...	1:1.63

The Chichoki Power House will be connected to the Main Rasul Grid at Sheikhupura and most of the Power will be utilized for the tubewell pumping and reclamation of the water logged areas in the Rachna Doab.

2. **Foundation Explorations.**

As the Power House structures are very heavy, it is a general practice all over the world to find a good rock foundation for building such structures. In certain cases if the rock foundation is not near the surface but can be found at a reasonable depth, it is usual to transmit the Power House load to the solid footing by driving piles In the plains of the Punjab, however, rocks are not encountered even at a depth of several hundred feet. It is, therefore, not a practicable proposition to go down to the rock level and as such it is necessary to find other ways and means of founding heavy structures. Good compact sand is an excellent medium for bearing heavy loads but to encounter such a uniform layer of sand in nature is impossible There are always admixtures of silt or clay or if the sand layer is free from such impurities, it is always found that there are certain clay or other soft soil lenses under-neath. In such cases, therefore, it is very necessary to make detailed and careful investigations before any venture of any magnitude is undertaken as otherwise there are very strong chances of failures of the foundation

In the case of Chichoki Power House detailed foundation investigations were carried out by a reputed firm Messrs Swiss Boring Overseas Corporation Limited. The field investigations consisted of the following :—

(a) Five exploratory borings at the proposed site of the Chichoki foundations. This also included disturbed and undisturbed sampling. The borings were executed by the percussion method using casings of $4\frac{1}{2}$" to 3" in diameter.

(b) Determination of the relative density of the sand in situ by standard penetration tests.

(c) Determination of the permeability of starta under investigations by means of large pumping tests.

The laboratory tests were carried out on the following basis :—

1. Sand :—

   (i) Grain size distribution.

   (ii) Permeability.

   (iii) Angle of internal friction.

2. Clay :—

   (i) Grain size distribution.

   (ii) Natural water content, compressibility, and voids ratio.

   (iii) Compressibility.

   (iv) Permeability.

   (v) Atterberg limits.

Although for power stations and other heavy industrial buildings, the spacing of exploratory boring is kept 50 to 60 feet, in both directions, it was not considered necessary in case of Chichoki Power House. The soil in this area is known to be quite uniform and spacing was therefore kept 90 feet and the results have justified the same. The boring was taken to a depth of 80 feet below the proposed bottom of the raft or about 115 feet, below natural surface level. According to the international practice, the boring is usually taken down to a depth of 1½ times, the minimum width of the foundation raft. The depth of 80 feet below the raft was, therefore, considered quite satisfactory in view of the tentative raft width of 46 feet. The results of the foundation investigations and the 5 bore holes are given in plate No. 6.

3. Analysis of Results Obtained by the Subsoil Investigation :—

A reference to plate No 6 will show that the top stratum of the surface with a thickness of about 10 feet, consists of compact silt, while sand has been encountered right up to R.L 600 This 80 feet thick sand stratum is not uniform but consists of various coarse to fine grain layers of sand including a few lenses of clay and at times a small percentage of Kankar. The sand was generally found to be clean and as the results of the standard penetration tests show, it can be classified as "medium to dense." An important clay layer below the sand stratum must be taken into account. This clay layer varies in thickness from 5 feet to 25 feet and it was found that the subsoil water under this clay layer was under artesian pressure with a static head of 10 feet above ground level. Except for the clay layer, the investigations show that the foundation conditions are more or less uniform and that it will be possible to lay the foundation raft quite safely subject, of course, to the long term settlement of the clay layer.

4. Allowable Bearing Pressure :—

From the results of the bearing tests and the analysis of the samples, the bearing capacities of the foundation soil can be easily calculated from the three formulae as given below :—

1. Rankine ... Ritter

$$q = \gamma . t\, \lambda\phi^2 + b/2 . \gamma . \sqrt{\lambda\phi} . (\lambda\phi^2 - 1)$$

2 Terzaghi ... Mohr

$$q = \gamma\, (t + b/2) \tan^4 (45 + \phi/2)$$

3. Prandtl ... Reissner

$$q = \gamma . t . \lambda\phi . e^{\pi \tan\phi}$$

Where γ = unit weight of sand

t = Depth of sheet pile boxing

$\gamma\phi = \tan (45 + \phi/2)$

ϕ = Angle of internal friction

b = Width of raft foundation

It will be seen from the above formulae that while the first two are based not only on the mechanical analysis of soil and its bearing capacity they take into consideration the dimensions of the foundation raft. The third formula does not take the dimensions into cognizance and is as such considered to be only an approximate and quick method of judging the bearing capacities It will be seen that the dimensions of the raft have a great bearing on the bearing capacities of the soils and a bigger raft will always be able to take more load on the same foundation as compared to a smaller raft. Messrs. Swiss Boring Company, therefore, calculated the ultimate and allowable bearing pressures according to these formulae. They were given the tentative dimensions of the raft as 184′ × 46′ and basing their calculations on the same their results are tabulated below :—

Table No. 1.

Formula used	Ultimate bearing pressure in tons/sq. ft.	Allowable bearing pressure in tons/sq. ft using factor of safety of 3.
1. Rankine——Ritter.	17.0	5.7
2. Terzaghi——Mohr.	13.2	4.9
3, Prandtl——Reissner.	12.4	4.1

The dimensions of the raft have now been changed to 184 feet × 120 feet. The line drawings received from the manufacturers make it imperative that the forebay should also be founded on the same raft as the machine room. The safe bearing capacities have therefore been recalculated and are summarised in the following table :—

Table II.

Formula used	Ultimate bearing pressure in tons/sq. ft.	Allowable bearing pressure in tons/sq. ft. using factor of safety of 3
1. Rankine—Ritter	35.1	11.7
2. Terzaghi—Mohr	24.9	8.3
3. Prandtl—Reissner	12.4	4.1

The comparison of the two tables will show that in spite of the change of the dimensions of the raft, the results obtained due to formula 3 are the same while in case of Rankine and Terzaghi formulae, the results vary greatly and are more in keeping with the actual conditions. Even if the lowest value obtained above is adopted, the safe bearing capacity of the soil comes to 4.1 tons per square foot.

On the other hand standard penetration tests carried out in each bore hole at every 5 feet intervals gave varied results and it is necessary to keep the same in view. Terzaghi has carried out extensive investigations in this direction and he has obtained certain empirical results correlating the standard penetration tests and the allowable bearing pressure of the soil. His results are tabulated below in table No. III.

Table No. III

Allowable bearing capacity for rafts on sand.

Relative density of sand.	Loose sand	Medium sand	Dense sand	V. dense sand
1. No. of penetration blows per ft.	Less than 10	10-30	30-50	over 50
2. Allowable bearing value in tons/sq. ft.	Less than 0.7	0.7-2.5	2.5-4.5	over 4.5

A reference to plate No. 6 will show the number of penetration blows obtained in various bore holes at the proposed raft level. These blows, vary from 25 to 60. A look at the charts will show that the lower figure is exceptional and mostly it is much higher than this. Messrs. Swiss Boring Corporation while making their recommendations have taken this lower figure for safety reasons and recommended a safe allowable bearing pressure of 2.5 tons per sq. ft. on the other hand it is clear that this low value is due to some isolated loose sand pocket which cannot be wide spread. The average number of blows at the raft foundation level comes to 41 which would place the safe bearing pressure at 3.6 tons per square foot which appears to be much more rational.

While making the foundation studies for Pathri Power House in India the same analysis was carried out by means of Rankine and Terzaghi formulae. The third formula was not referred to as it was considered inadequate for the purposes of determination of safe bearing capacity of soil. In the case of Chichoki Power House also the Prandtl formula has been discarded as being only approximate and the bearing capacity of the foundation soil has been calculated from the other methods as described above.

5. Permeability of Sub-Soil Strata.

The permeability co-efficient was determined by the standard method. A 22" diameter casing pipe was sunk by percussion boring up to a depth of 77 feet below the ground level which is about 7 feet above the clay layer. A strainer of 9" diameter was lowered into the casing pipe and the space between the two was shrouded by gravel and coarse sand and then the casing pipe withdrawn. Six piezometeric pipes of $1\frac{1}{2}$" diameter were sunk in the two principal directions to note the lowering of the ground water level. Two different discharges of 2.64 gallons and 4.12 gallons per second were used for the purposes of the test. It was found that the co-efficient of permeability worked out to 2×10^{-2} inches per second along the axis of Power House and 2.7×10^{-2} inches per second in the direction at right angles to it. The variation in the above values is due to different stratification of the sub-soil in the two directions. The average value was taken as 0.002 feet per second.

6. Dewatering Arrangements.

The Irrigation Research Institute calculated the expected discharges from the Power House pit by basing their calculations on the above value of co-efficient of permeability. They assumed that the Power House pit could be affected by both Upper Chenab Canal on the one side and sub-soil water on the other. Calculations were made by using Darcey's equation and it was found that the total expected discharge both from the canal and the sub-soil reservoir would come to 10.67 cusecs.

According to the above report, it was suggested that a percolation equal to about 15 cusecs could be expected in the excavated area for which 10 tube

wells of 2 cusecs capacity each would be sufficient. The report also suggested providing 4 sets of 2 cusecs well point strainers in the corners of the raft for pumping the pores water etc. Actually 10 submersible pumps have been arranged around the Power House pit as shown in fig: No. I.

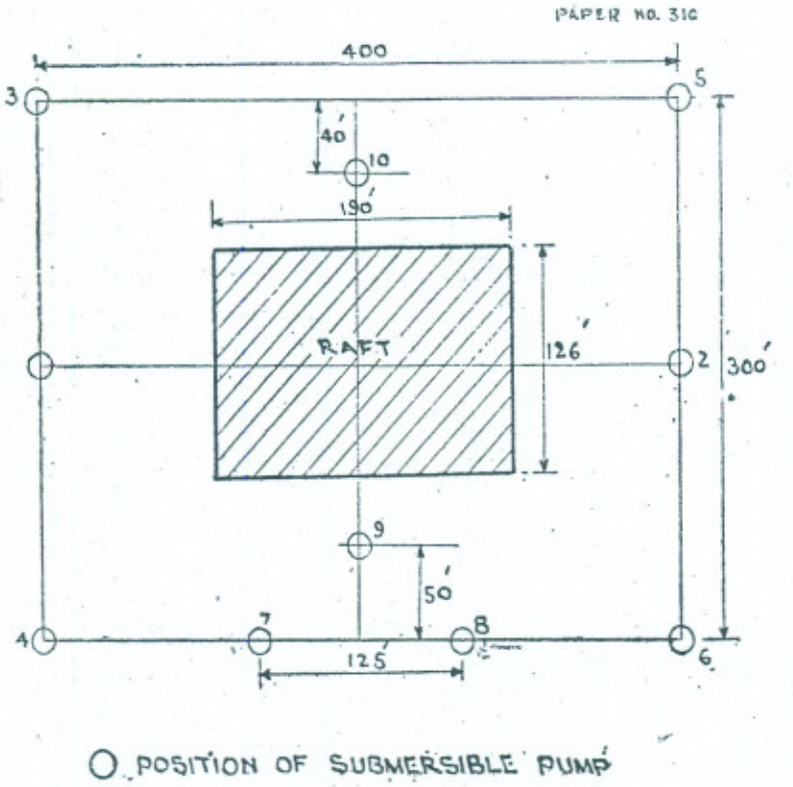


Fig. No I.

No well point strainers have been provided at site but the actual working of these submersible pumps have shown that there is no need for any extra well points as the sub-soil water level is well below the foundation level.

The use of this method of dewatering has not only provided an absolutely dry working space but also a well compacted foundation base for the Power House raft. This method, which has been used for the first time in the Punjab Irrigation Branch, has proved to be very successful as it has done away with all the messy slush work, and increased the progress of earth work manifold. In the prevalent method of open pumping through sump holes, a number of drains have to be provided and the water is always fully charged with fine sand particles thus disturbing the foundation base and making it unfit for any heavy loads. In the case of such heavy structures as power houses, such chances cannot be taken and hence a resort to the tubewells.

7. Settlement of Foundations.

Due to the presence of a clay layer under the foundation, long term settlements are naturally to be expected. When a load is applied, a change in the volume of voids occurs and water is squeezed out. As the clays have a very low permeability, the passage of this water is very slow and the settlement therefore takes a very long time. In case of sands, the change in the volume of voids is quick as the water can be displaced quickly due to a high co-efficient of permeability. But in the case of clays, this is not possible and the settlement of the foundation continues long after the loads have been applied and as such great care has to be exercised in the design of such foundations. Due to the variation in the thickness of the clay layer, a differential settlement is also anticipated and has been very carefully calculated, and every effort has been made to keep it within the allowable limits. The results of the settlement analysis show that the maximum differential settlement will occur between the edge of the raft and its centre and will be of the order of 1.3" which is well within the allowable limits. It is expected that the R. C. C. mat will take up the settlement gradually and there will be no appreciable effect on the superstructure. The process of consolidation in the clay layer is expected to continue for at least six years after the load has been applied, after which it is expected that the foundation will become stable.

8. Separate Foundations for Control Block and Machine Room.

In order to keep the actual loads as low as possible it was proposed to build the machine room, control block and the forebay structures on the same foundation. The main advantage in adopting this proposal is obviously the smaller intensity of pressure that would be transmitted to the foundation soil on account of the large size of the raft. But on the other hand this proposal would have resulted in not only a very thick raft for the control block but also a very heavy sub-structure for transmitting the control block loads to the foundation raft. The cost in this case would also be prohibitive. Besides, due to the earth filling on the raft under the control block, the advantages gained in reduction of the load would also be very meagre and inconsistent with the extra costs involved.

It was, therefore, decided to construct two different foundation rafts for the control block and the machine room cum-forebay structure (Plate No. 3.) The two rafts will be at different levels as required according to site conditions with a straight and clear joint through out the height and breadth of the power house. The present design is based on this latter proposal and a clear joint has been provided as advocated by Mr. S. S. Kirmani in his paper "The Design and construction of Irrigation Structures" in the P.E.C. Session 1954. This joint will provide for the differential settlement between the two structures and avoid any unsightly cracks in the floors and walls as experienced in Rasul Power House.

9. Actual Load Conditions :—

A summary of the dead and live loads for various portions of the power house are given below :—

Load of super-structure	...	1520	tons
Load of Machinery	...	880	tons
Live load	...	920	tons
Load of sub-structure, including water in the passages	...	47380	tons
Load of raft	...	8900	tons
Total	...	59600	tons
Size of raft	...	120×184	
Average intensity of pressure as per above	...	$\frac{59600}{120 \times 184}$ = 2.7	tons per sq ft.

In spite of the fact that the super-structure has been designed with the lightest possible materials, it has not been possible to decrease the above intensity of pressure any further. Actually the load of the sub-structure and the raft alone, brings the intensity of pressure up to the allowable limit as prescribed by Messrs. Swiss Boaring Corporation. On the other hand, the excess of 0.2 tons per square foot, is too meagre to be of any effect specially when the loads have been calculated on a very liberal scale.

10. Effect of Vibrating Machinery on Loads.

The periodic impulses produced by the rotating parts in a Power House cause a series of vibrations in the Power House structure. At times it happens that the frequency of vibrations coincides with the natural frequency of the soil foundation system thus creating resonance and readjustment of the soil particles. This readjustment is very dangerous as it may result in appreciable compaction of the soil and hence settlement of the structure. In order to eliminate its danger, it is always necessary to make a detailed analysis of this aspect before any actual work is taken up. In the case of power houses, the viberations not only take place due to the movement of turbine runner but also in case of trips when-ever a fault occurs in the system. In such a case there is a sudden racing of the machines beyond their designed speed and then with the working of the protective system, the machines are tripped. This results in a very severe jolt to the whole power house structure and high frequency vibrations are set up all over. The trips are fortunately a rare occurrence and the time interval involved is infinitesimal. The actual effect on the foundations is, therefore, negligible and can be safely discounted.

In case the frequency of the vibrating parts comes within the natural frequency of the foundation system, it is necessary to provide an elastic layer between the foundation and the structure. In the case of Chichoki Power House, however, the frequency of the turbines of the turbo-alternator sets is 107 R P.M. which is far below the optimum range for sand paricles, i.e., between 500 to 2500 impulses per minute. It is, therefore, obvious that no excessive settlements will occur due to this affect at Chichoki.

The inertia forces, however, will increase the pressure on the foundations and their effect will, therefore, have to be taken into consideration. The total vertical disturbing force is given by the following formula :—

$$P = Q \times \frac{1 + \frac{f_2}{f_1}}{1 - \frac{(f_2)^2}{(f_1)^2}}$$

Where Q is the weight of the rotating parts of the machine, f_2 is the frequency of turbines and f_1 is the natural frequency of the soil foundation system. In order to calculate the natural frequency of the soil foundation system (f_1) the following formula has been used :—

$$f1 = \frac{1}{2\,II} \sqrt{\frac{cs\ g}{w_1 + w_2}}$$

where cs = Spring's constant.

w_1 = Weight of machine and the structure.

w_2 = Weight of body of soil which participates in the vibations.

g = 32.2 feet per second.

Spring's constant, however, is calculated as below :—

CS = A Ds

Where A = Area of raft.

Ds = Co-efficient of dynamic sub-grade which in turn is calculated from the relation $ds \frac{(cm)^{-1}}{q}$ which ranges between 2 for loose sand and about 3 for dense sand ;

q being the bearing capacity of soil.

Working on the above basis f_2 comes to 1.78 revolutions per second and f_1 to 7.4 revolutions per second. As f_2 is less than f_1, there is no possibility of resonance and hence settlement of foundation.

As the effect of the vibrating loads on settlements is greater than the static load, the equivalent static load has been taken as three times the vibrating load for design purposes. This static load when distributed over the entire raft area gives the increase in intensity of pressure on soils due to vibrations. In case of Chichoki, it has been calculated as 0.07 tons per square foot.

11. Earthquake Effect :—

Earthquakes cause both horizontal and vertical ground movements. The former is many times the vertical motion which is so small that it is not usually taken into account. The following formula is used for calculating the magnitude of the lateral earthquake forces :—

where $F = C (W_D + KW_L)$

F = Lateral earthquake force.

W_D = Dead Load.

W_L = Live load.

K = Constant depending on nature of live load.

C = A constant depending upon nature and situation of site according to earthquake zones.

Pakistan metereological Department has laid down that for the Lahore area value of C should be taken as ranging between 0.1 to 0.05. In the present case a value equal to 0.083 has been taken. The lateral forces have been calculated at the roof and floor levels and at the centre of gravity of the remaining substructure. Thus values of the three lateral forces f1, f2, and f3 (figure No. II) have been determined. The maximum intensity of pressure on foundation soil developed due to earthquake effect is calculated from the following equation :—

$$F = M/Z$$

where M = Total moment of the lateral earthquake forces F1, F2, and F3 etc., about the bottom of raft

Z = Modulus of section of the raft.

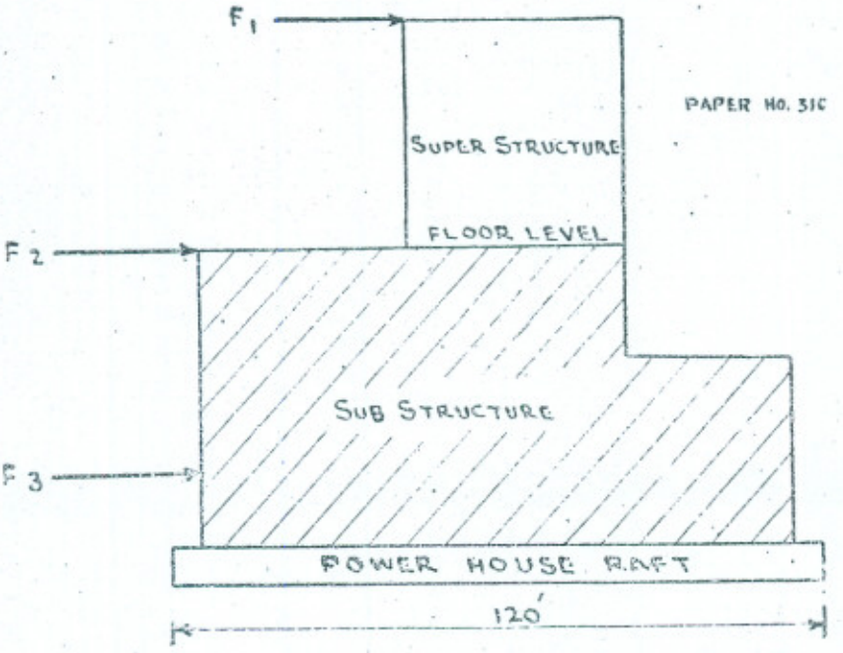


FIG. NO. II

In case of Chichoki machine room raft, the maximum intensity of pressure of soil due to the earthquake effect works to 0.35 tons per square foot. As the maximum stress on the soil will develop only when the earthquake vibrations move along the smaller dimensions of the raft, the calculations have been carried out in this direction only.

12. Wind Effects :—

The London County Council in its by laws has laid down that the effect of wind in the increase of loading can be easily neglected if the height of the building is less than twice its width. The height of Chichoki Power House being only 100 feet and much less than twice the width of the raft ($120 \times 2 = 240$), this effect has not been considered in the design of the foundations.

The final intensity of pressure on foundations are summarised below :—

Average intensity of dead and live load	...	2.7 tons per sq. foot.
Intensity due to vibrations.	...	0.07 ton per sq. foot.
Intensity due to earthquake forces.	...	0.35 ton per sq. foot.
Total pressure.	...	3.12 tons per sq. foot.

The actual load conditions show that the average load of 2.7 tons/sq ft. is about 0.2 ton per square foot in excess of that recommended by M/s Swissboring Corporation. A reference to tables II & III will show that the recommendations by the Company are rather, on the conservative side. The actual load of the over burden at the foundation raft level comes to 1.7 tons per square foot which leaves only 1.0 ton per square foot as the extra load to be borne by the soil. This much load is even safe for most of the top soils in the Punjab. The average load intensity of 2.7 tons per sq. foot calculated for Chichoki is therefore, quite safe and presents no danger. The vibration load of 0.07 ton per square foot is also too small to be of any real effect, while the earthquake loads, which are quite substantial (0.35 tons per square foot), fortunately occur only rarely and the high ultimate safety factor of $\frac{24.9}{2.7} = 9.22$ (Terzaghi – table II) can easily take care of the same. Even the maximum point load of 3 87 tons per square foot (plate No. 7) on the raft is well on the safe side; safety factor being of the order of $24.9/3.87 = 6.4$.

13. Design Aspects :—

The machine room foundation raft has been designed as a R.C.C. structure boxed on all sides by 20 feet deep steel sheetpiles The raft will have a dimension of 120′ × 184′ and will be quite independent of the sheet piles so as to allow the raft to take settlements without restraint from sheet piles (plate No 2 & 3), The joint between the sheet piles and the raft has been kept as 1½″ thick. In order to prevent sand blowing through the joint,

two layers of copper flashings will be provided and as an additional precaution a 3" coir rope impregnated with bitumen will also be incorporated in the joint. An inverted filter is also being provided at the bottom so as to prevent any possibility of sand going through. The joint has, therefore, been made fool-proof as its failure will disturb the foundation soil which may result in greater differential settlements, thus causing ultimate failure of the whole structure, (Plate 4).

The sheet pile will have an R.C.C. coping of 5 feet width so as to cover any displacement of top of pile during the driving operations. The raft will be laid at an elevation of 651.6 with a minimum thickness of 6 feet and will have a flat bottom. The length of the raft along axis of turbe-alternators will be 184′ – 0″ and along right angles to it 120′ – 0″

Theoretically there should not be any pacca floor downstream of draft tubes, as the whole of the energy of water would be imparted to the turbine blades, and water should come out calmly through the draft tubes. But at the Rasul Power Station it has been seen that there is considerable turbulance in water as it emerges out from the draft tube. This is due to defective designing of the turbines and there is always some residual energy in the water coming out of the draft tube exits and must therefore be guarded against.

It has, therefore, been decided to keep 40 ft long pacca floor beyond the draft tubes. This floor will also be boxed by means of 20 ft. deep sheet piles and has been designed for the full hydrostatic pressure, that is supposed to act on the floor.

In order to distribute the load as uniformaly as possible on the raft and prevent unequal settlements the centre line of the turbines (the seat of C.G. of loading) will be kept nearly in the centre of the floor i.e., 58′ – 5″ from the upstream edge of the raft and 61′ – 7″ from the downstream edge of raft.

There was a proposal of providing separate foundations for the intake, the central portion of Power House and the draft tube structures keeping a clear joint between them, in view of the uniformity of loading on these portions of the Power House, However, due to following drawbacks this proposal had to be dropped.

1. The intensity of loading in the central portion was found to be maximum and was well beyond the safe limit set by the field tests.

2. The unequal settlements of the foundations of intake, machine room proper and draft tube structures on account of difference in loadings, would have disturbed the intake and draft tube settings. This would have resulted not only in water turbulence but also in severe damage to the joints. In order to decrease the intensity of loading on foundation soil and to

keep the centre line of turbines in the centre of the raft, it was therefore decided to keep a single raft under the intake, machine room block and the draft tubes. If the centre of gravity of the load is at the centre of the raft and the subsoil profile is such that the raft rests on the same type of stratum under its whole area, the settlement will be uniform the base of raft will remain horizontal and magnitude of settlement will materially cause no damage.

Pattern on loading on raft

The exact determination of the load pattern on raft is difficult on account of great complexity. of structure. An approximate load pattern has, therefore, been determined on the following assumptions.

1. The load distribution is uniform along the axis of the Power House.

2. The load distribution varies at right angles to the axis of Power House.

For computing the load distribution on raft the area of raft under one unit has been divided in four parts.

1. Area above upstream wall of Power House.
2. Area between upstream wall of Power House and centre line of the turbine.
3. Area between centre line of turbine and downstream wall of Power House.
4. Area beyond downstream wall of Power House.

The variation in the dead and live load intensity has therefore, been plotted at right angles to the axis of the power house and is shown in plate No, 7.

The machine room foundation raft has been designed as a plain slab raft, reinforced with steel bars both at top and bottom. This type of raft is most suitable for very irregular loadings, such as in the Chichoki power house and for structures founded on soils where there are possibilities of differential settlements. The slab has been designed for the full load of the Power House distributed over the raft, without any deduction for hydrostatic uplift or foundation excavations plus the pressure intensities caused by earth-quake and vibration of the machines.

Bending Moments :

For calculating bending moments the following procedure has been adopted.

1. Below the draft tube gates :—

The span considered is the clear distance between the two main piers plus the width of the intermediate draft tube pier, as this pier has been cut by the draft tube gates and has very little superimposed load. The effect of intermediate pier, however, has been taken as a concentrated load at the middle of span, while calculating bending moment on this part of the raft. This span is the most heavily reinforced.

2. Above the draft tube gates :

As the intermediate pier will be taking substantial load of the Power House structure in this region, it has been considered to be acting as a support, and the effective span has been taken as the clear distance between the divide wall and the intermediate pier.

3. The divide walls and side abutments downstream of the draft tube gates will have to resist the total soil reaction from below which is far more than their superimposed load. They have, therefore, been designed as cantilevers projecting from the down-stream wall of the Power House.

4. The load of the intake structure will be transmitted to the raft by means of two cross walls running parallel to the axis of the Power House and similar walls under the forebay piers and abutments. One of the cross walls will be under the intake gates and the other just at the beginning of the intake chamber. These chambers thus formed will be kept hollow so as to decrease the loading on foundation soil. As the spans of these chambers are almost equal to each other both way reinforcement has been designed for this portion of raft.

5. Cantilever Portion :—

The R.C.C. raft will Project at some portions beyond the power house structure for the following three reasons :—

(a) For decreasing the intensity of loading on foundation soil.

(b) For laying a rectangular raft which is strong and requires no special junction sheet piles at the corners.

(c) For a thorough anchoring of the foundation raft by providing an adequate earth cover.

The span of the cantilever considered is the lenght of raft projecting beyond the edge of the Power House wall, and it has been designed for the total pressure acting from below, deducting the weight of concrete of raft and the earth filling upto the ground surface. The earth filling will be carried out simultaneously with the raising of the Power House structure so as to avoid excessive stresses and consequent damage to the foundation.

Reinforcement :—

Equal and continuous reinforcement have been provided both at top and bottom as there is always the danger of differential settlements which results in the increase of magnitude of stresses. The amount of reinforcement calculated has been increased by 50% since the difference between the theoretical and real bending moments can be quite large on account of uncertain behaviour of foundation soil. (Terzaghi & Peck—soil mechanics) The reinforcement of raft downstream of draft tube gates has worked out to 90 lb rails 8" to 12" apart on account of large span considered (43 ft.). The remaining portion of raft will be reinforced with $1\frac{1}{2}$" dia bars. Cross sections of the raft showing detail of reinforcement are given in plate No. 4.

Shear Reinforcement :

Shear reinforcement has been provided where the shear stress has exceeded 75 lbs/□", in the form of 7/8" or 1" dia. continuous stirrups as shown in plate No. 4. In the portion of slab d/s of draft tube gates shear stress has worked out as high as 150 lbs/□" and so shear reinforcement has been provided in the form of a continuous stirrup 1" dia at 7" spacing while towards the centre of span, spacing has been increased to 13".

In the portion of slab u/s of the draft tube gates, the shear stress has worked out to 92 lbs/□". To strengthen the slab against shear 7/8" dia bar at 7" spacing has been provided near the supports and spacing has been increased 19" near the centre of the span.

Similarly the shear stress in the slab below intake chamber has worked out to 101 lbs/□" and 7/8" dia bars at $6\frac{1}{2}$" centres near the support has been provided, the spacing has been increased to 17" near the centre of the span where shear is minimum.

Construction Joints :—

The construction joints have been provided on the consideration that 5,000 cft. of concrete will be poured in a single shift. Two concrete mixes will be adopted for machine room raft 1:$1\frac{1}{2}$:3 and 1:2:4; the former will be used for $1\frac{1}{2}$ feet of top and bottom concrete where there is a heavy concentration of reinforcements. The latter mix will be used for the middle portion as shown in plate No. 5.

Theoretically the construction joints should be provided at the position of minimum shear i.e , near the centre of the span but there is a possibility of the deflection of raft at right angles to the axis of Power House which might open out any joint placed parallel to the axis, allowing the water to rise, thereby damaging the reinforcements. The joints in the three layers have, therefore, been staggered. In view of the staggering all the joints cannot be placed at the point of minimum shear and as such some of the joints have been provided near the points of contra-flexure.

SUMMARY AND CONCLUSIONS

The purpose of presenting this paper is to give a clear idea to the Engineers at large who will, in future, work on the design of foundation structures of Power Houses on alluvial soils. Such Power Houses are proposed to be constructed in Pakistan at Gujranwala, Mangla, and Khanki etc. Chichoki Power Station which is under construction at present, is probably the first Power House of its kind whose Civil Works will be designed and constructed by Pakistani Engineers.

The paper in addition to a brief description of the Power Station discusses in detail the foundation investigations carried out at site. The field investigations consisted of the following :—

1. Exploratory borings.
2. Standard penetration tests for determining relative density of sand.
3. Pumping test for determining the co-efficient of permeability of sub soil strata.

As a result of the field investigations and tests carried out in the Laboratory, the bearing capacity of foundation soil and capacity of pumping equipment required for excavation of Power House pit have been determined.

Actual load conditions and the method of computing the increase of load intensities on account of vibrations of the machinery and siesmic forces have also been discussed in the paper.

The design of machine room raft has been discussed in detail. The reasons for not providing a single raft foundation under the machine room and the control block have been elucidated and the various advantages in providing a continuous foundation under the machine room, forebay and draft tube structures have been given in detail.

Acknowledgements :—The authors wish to acknowledge with thanks the valuable advice given by Mr. A. D. Ashraf Chief Engineer and Secy. to Govt. Punjab under whose benevolent guidance the actual design was carried out. Thanks are also due to M/s. Irshad Hussain, Research Officer, B & R Research Laboratory and S. M. Niaz, D.C.D. (Links) for their valuable suggestions in the preparation of the paper.

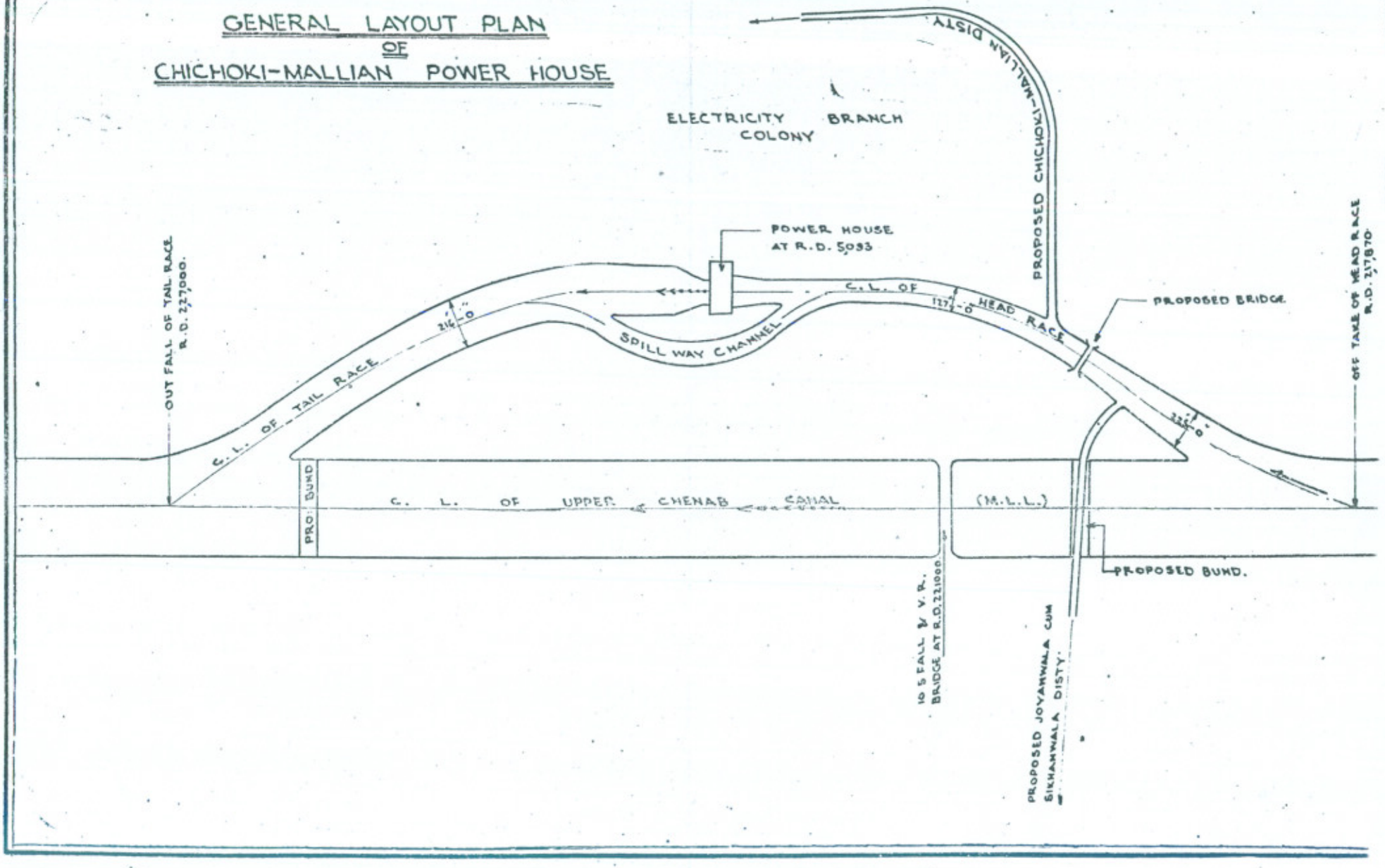
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PAPER NO. 316

PLATE NO.

GENERAL LAYOUT PLAN OF CHICHOKI-MALLIAN POWER HOUSE



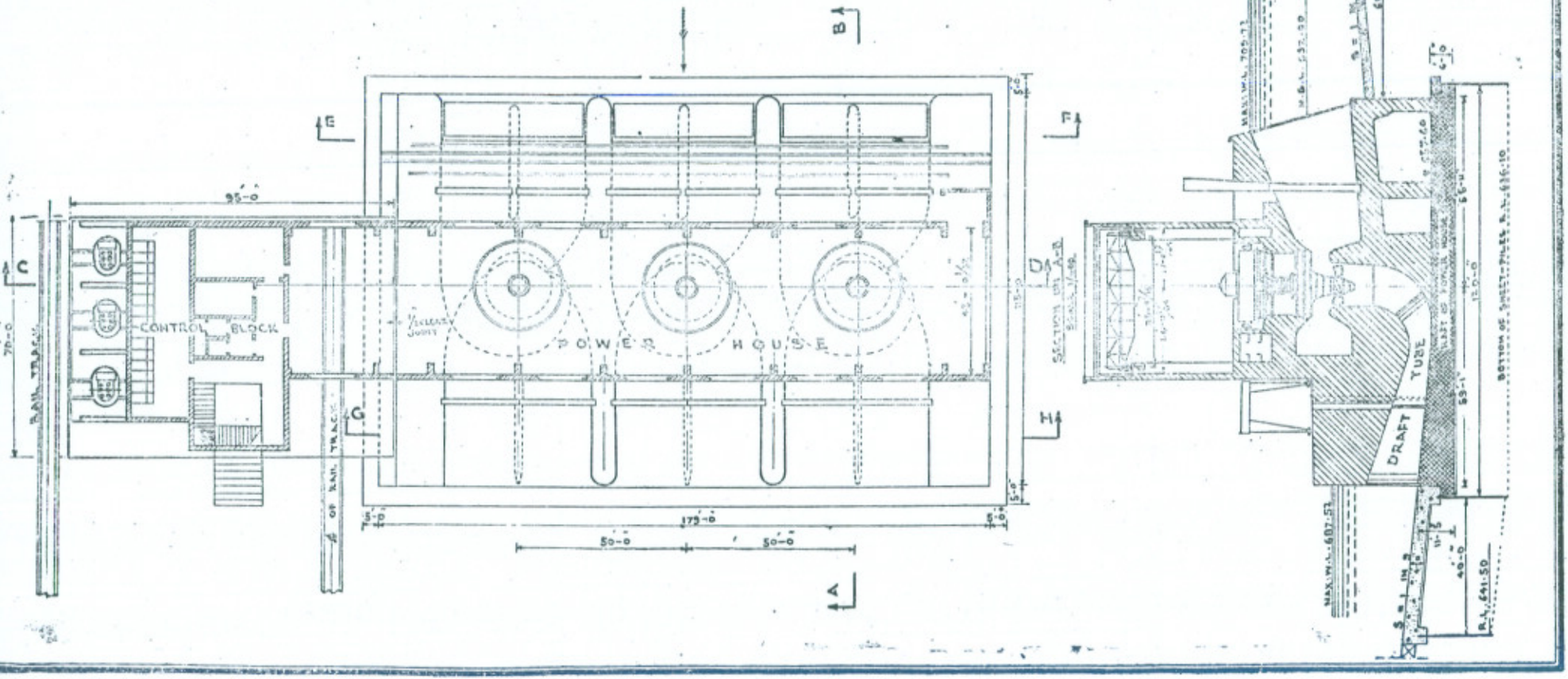
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PLATE NO:- 2

CHICHOKI - MALLIAN HYDEL PROJECT

PLAN OF POWER HOUSE & CONTROL BLOCK

SCALE = 1/400

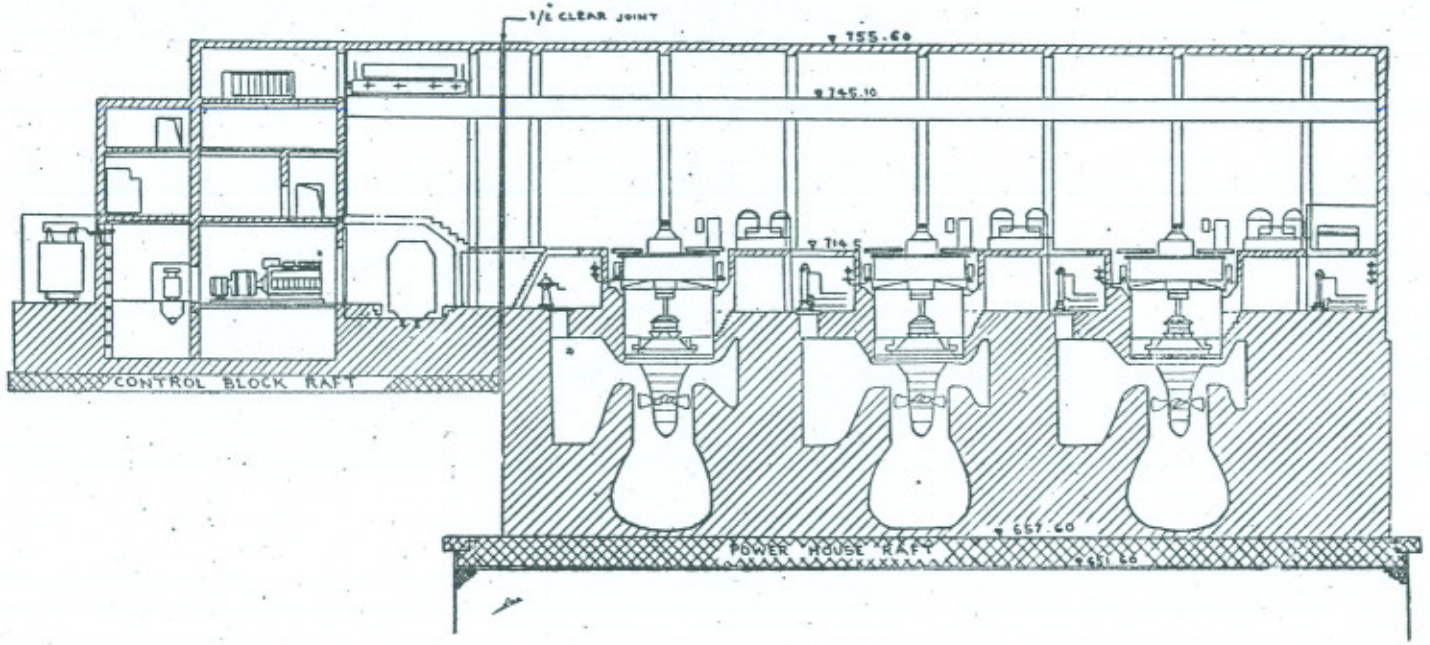


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SECTION ON C.D.

SCALE 1/400

PLATE NO. 3



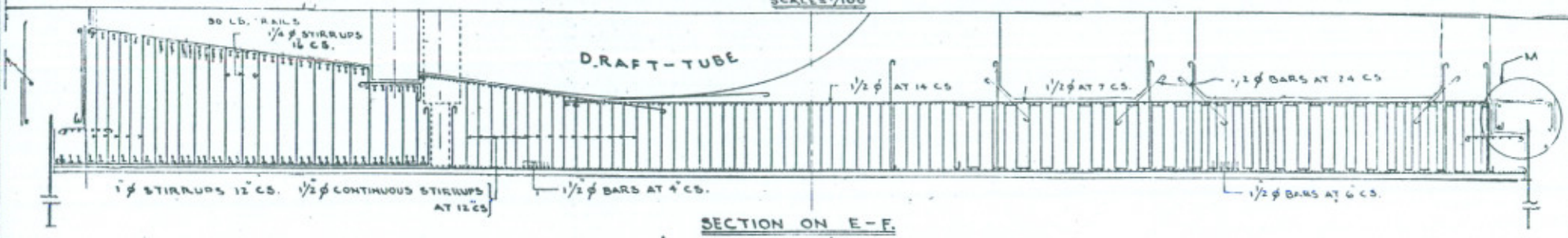
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PLATE NO:- 4

REINFORCEMENT IN POWER HOUSE RAFT

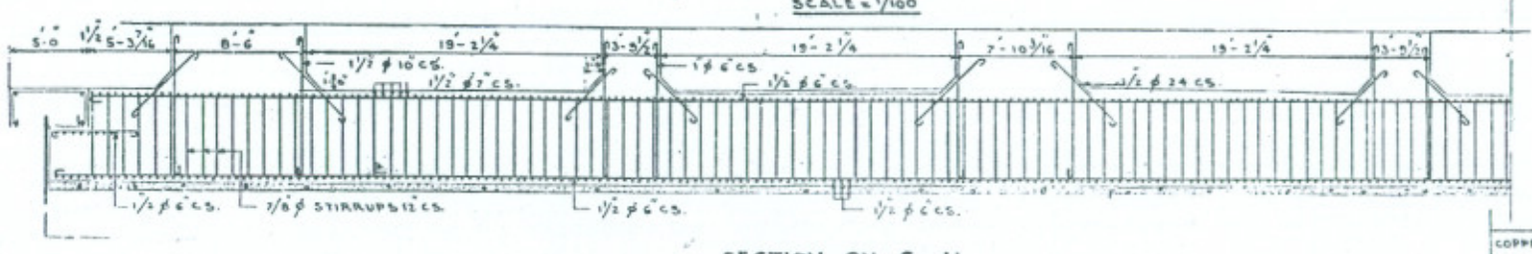
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SCALE = 1/100



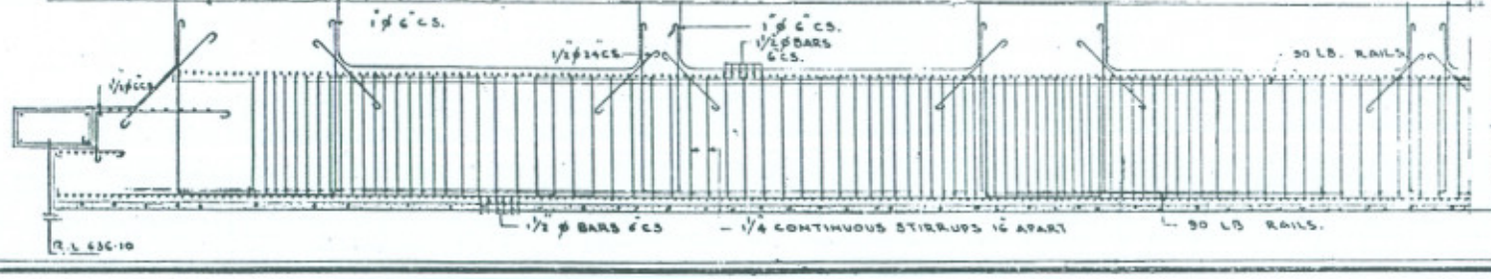
SECTION ON E-F.

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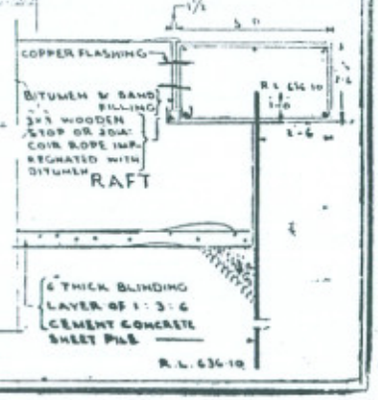
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SCALE = 1/100



DETAIL AT "M"

SCALE = 1/50

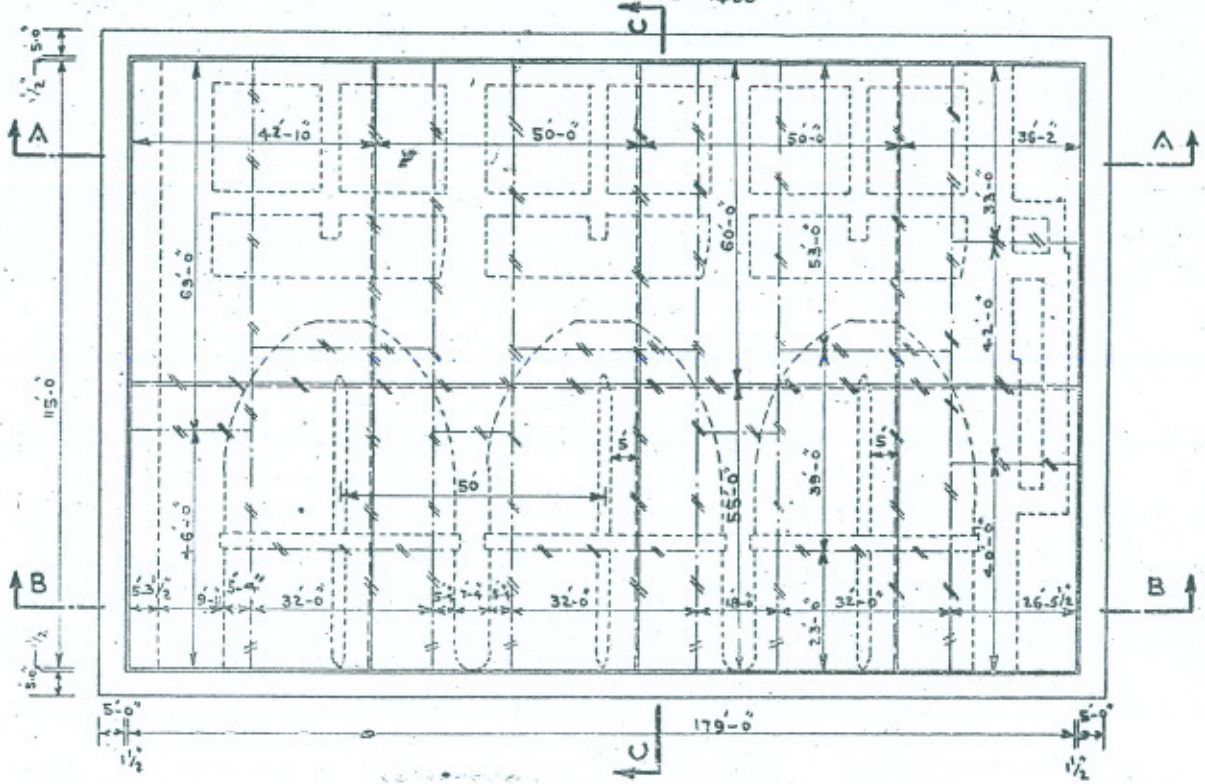


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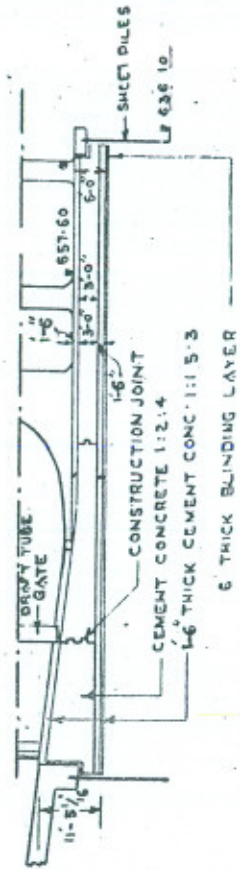
PLATE NO:- 5

PLAN SHOWING CONSTRUCTION JOINTS IN POWER HOUSE RAFT

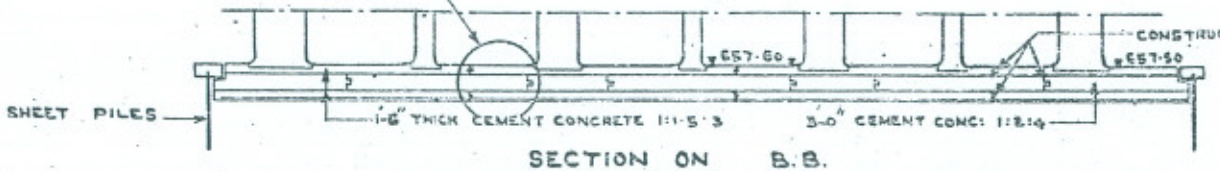
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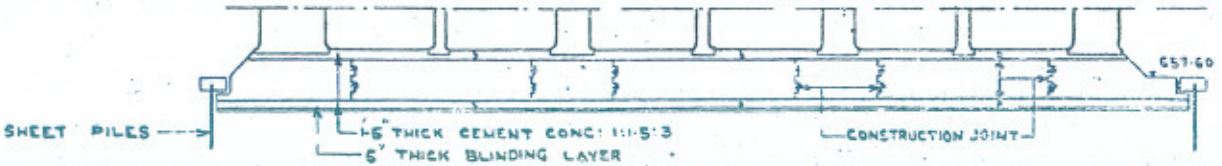
SECTION ON C.C.



'A' SECTION ON A.A.

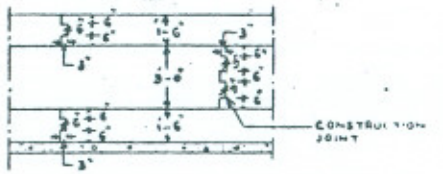


SECTION ON B.B.



DETAIL AT 'A'

SCALE = 1/100



NOTE:-

CONSTRUCTION JOINTS IN TOP AND BOTTOM CONCRETE SHOWN THUS

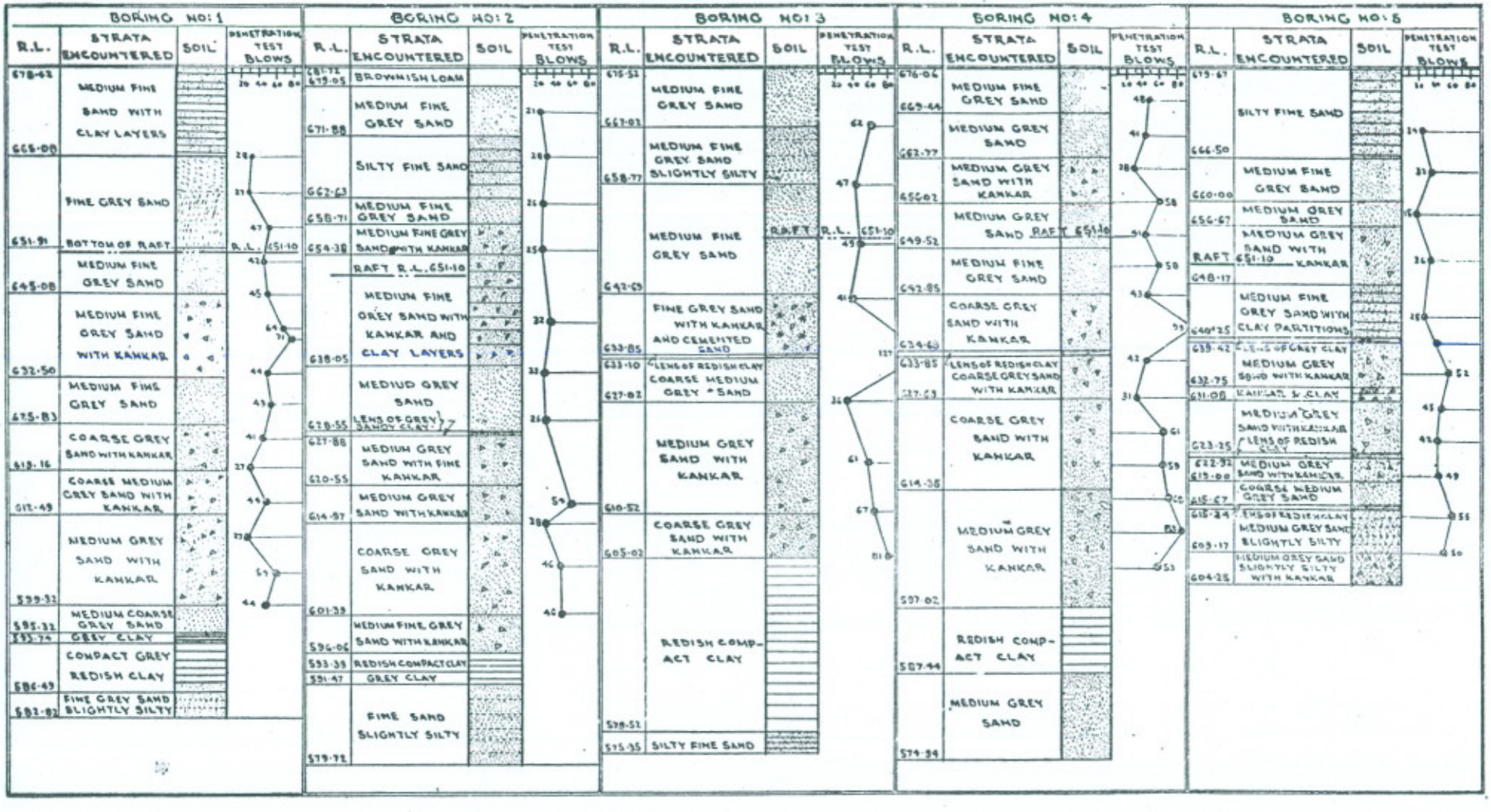
CONSTRUCTION JOINTS IN THE MIDDLE CONCRETE SHOWN THUS

BY: M. SADIQ

PAPER NO. 316

CHICHOKI MALLIAN POWER HOUSE

PLATE NO. 6



PAPER NO. 316

PLATE NO- 7

GRAPH SHOWING

DEAD & LIVE LOAD INTENSITY OF PRESSURE BELOW RAFT

