PAPER No. 177.

THE CONSTRUCTION OF A REINFORCED CONCRETE BRIDGE ON THE GRAND TRUNK ROAD OVER THE BHIMBER NALLAH, IN GUJRAT DISTRICT, ON VIBRO-PILE FOUNDATIONS.

By P. L. Varma, Assistant Engineer.

A number of bridges, on the Grand Trunk Road, built in the early sixties of the last century, consisted of timber structures carried on wooden piles. Most of these rivers being non-perennial, the piles and the bottom parts of the structures were exposed to alternate wetting and drying, with the result that, due to decay thus set up in the timber, these structures had nearly spent their useful life, and some of the more important ones, had to be replaced by modern structures built in more permanent materials and designed to carry heavier and faster load units.

To cheapen the cost of construction, reinforced concrete pile foundations were first employed in the construction of the Chel Bridge situated in mile 218 of the Grand Trunk Road in Attock District. The sub-soil consisted of hard clay, and innumerable difficulties were met with in driving the pre-cast 14″ hexagonal piles. The driving was so heavy that the piles could not stand the steam-hammer blows, and the first couple of piles cracked at various points and their heads invariably got so much shattered that these piles had to be taken out and rejected. All attempts to drive the piles successfully having failed, they were put down by first sinking an 18″ diameter steel pipe to proper depth, lowering the pile into the same and driving it to refusal, allowing 24 hours rest and redriving it to refusal. The method, although quite successful, proved expensive and very tedious.

At about this time, the Royal Engineers of the M. E. S. in the Frontier Province, were adopting Vibro-piles for foundations, and had very successfully developed their use in flowing water. The bridges they put up were similar in design to that at Chel, *viz.* :—single row pile-bents connected by reinforced concrete transoms which carried the superstructure. It was at their suggestion, therefore, that Vibro-piles were used in the foundations of the Wah Bridge in mile 198 of the Grand Trunk Road, near Hasanabdal ; and their successful use there and in the foundations of the shingle-bedded rivers in the Frontier Province led to their use in the clay bed of the Bhimber Nallah at mile 74 of the Grand Trunk Road near Gujrat, where, however, a new experience was gained in their behaviour which is described later on.

Reinforced concrete piles are being extensively used for foundations of all types in these days. Big buildings, large bridges, and heavy machinery are all being successfully carried on pile foundations. The ease and quickness of executing th e job, together with the certainty of

results, has led to their extensive use ; and the use of piles, cast *in situ*, is coming more and more to the forefront for the following reasons :—

(1) The danger arising from a cracked pile is eliminated.

(2) In most of the piles cast *in situ*, the surface friction is slightly greater than in the case of smooth pre-cast piles.

(3) As a steel tube is used for driving, the piles can be cast in shingle bedded rivers, where the driving of pre-cast piles would be impossible.

In the 'Vibro' system of driving, the piles are cast *in situ*. As some may be unfamiliar with this system, a detailed description of the same has been given in this paper, together with that of the modifications required in the case of varying strata encountered. Captain M. R. Jefferis, R.E., has developed a method of thickening the piles on top and driving them in water, which removes the great handicap to which piles, cast *in situ*, have been liable. This new system has also been described in the paper, and precautions necessary in using such piles in clay soils, based on the experience of such a soil at the Bhimber Bridge (a brief description of which is given) has been discussed.

'Vibro' Piling System.

The Plant.—A picture of the plant is shown in Fig. 2. The plant, in its essentials, consists of a wooden jib, a 3 ton winch and a boiler, a steam-hammer and a steel tube, carried on an I-beam frame which rests on rollers.

The jib or the driving frame is made of seasoned wood with all the joints bolted with gusset plates. It is mounted on the chassis with the help of a swivel pin, marked S, in the picture (Fig. 3) so that the jib can be erected in a horizontal position, and then hoisted up with the help of shear legs (shown in Fig. 4). The frame is capable of being swung back, as shown in the picture (Fig. 7) (which can be done by simply removing the wooden wedges, W, on the back) ; and the piles can be driven at a batter of 1 : 5.

In front run two channels (called leaders) all the way down the jib. They act as guides for the tube and the hammer, restricting the motion of the tube, and, consequently, the casting of the pile in the direction of the jib, which can be maintained in a vertical position.

The tube is usually of 16″ external diameter and $\frac{3}{4}$″ thick. The top consists of a specially formed head with a filling hole on the side to allow the concrete to be poured in. It is provided with a roller guide, running along the leaders which insures a straight drive. To absorb the blow of the hammer and protect the tube head from damage, the top is recessed slightly to take up a wooden dolly. The bottom of the tube is bulged to 17″ thus giving a 17″ pile. (Fig. 5).

The Hammer.—B.S.P. Single-acting Semi-automatic steam pile hammer is used. A view of the hammer and a section through the same is given in Fig. 6. The principle of such a hammer is that the

cylinder delivers the blow, and not the piston. Steam being led to the point from the boiler, through a flexible hose, is admitted into the cylinder through the hollow piston rod, P-R with the help of a piston valve.

The port holes are bored from the top of the piston, and as the steam enters it lifts the cylinder. As soon as the proper height is attained, the steam is opened to the atmosphere and the cylinder drops down under gravity. The stroke of these hammers is usually 4′—6″ ; and the weights vary from 1 ton to 3 tons. The same hammer is used for extracting the pile, when, as it rises, it strikes the upper end of a pair of links connected to the tube head and gives the necessary blow in an upward direction (Fig. 7). This hammer gives better results than the ordinary steam hammer, in which case, although the number of blows are large, they are absorbed by the pile and consequently are useless for the driving of a heavy tube or heavy piles.

A pair of links which are used for lifting the hammer or the pile, a skip for lifting concrete into the tube, and a few sheaves, mounted for various purposes, complete the plant.

Casting a Pile.

The operation of casting piles is very simple. A cast iron shoe is placed on the spot where the pile is to be driven (Fig. 5). The tube is brought to sit on it, the joint being rendered water-tight by using hemp smeared with white lead. The links carrying the tube and the hammer (shown in Fig. 7) are disconnected, and the tube driven into the ground by successive blows of the hammer.

The steel tube being able to stand heavy blows, can penetrate even boulder-bedded rivers. On encountering a very heavy boulder occasionally, the tube and the boulder both yield slightly and any displacement of the tube from the vertical position is set right by moving the jib in a direction opposite to the displacement.

A hole is thus formed in the soil, and as soon as the tube has gone to the proper depth, which is governed by the set (*i.e.*, penetration per stroke of the hammer) and not by the length of the pile, the hammer is lifted up with the help of the links, and the reinforcement cage lowered into the tube. Mixed cement concrete is then poured from top with the help of the skip (Fig. 9) and links connected to the pile. The hammer, as it is lifted, strikes the top of the links and raises the tube by $1\frac{1}{2}$″. When falling it drives the tube about an inch ; and as the tube rises by a sudden blow, a portion of concrete drops down from the tube. On the downward stroke, the same concrete is tamped into the hole in the soil. The shoe is left at the bottom. It will be seen that as the tube is constantly vibrating, the concrete to be used should be fairly dry so as to avoid seggregation of the component materials.

Thus a pile is cast in the ground whose load-carrying capacity will depend on the soil conditions met with. It will be seen, that as the concrete is laid green, the use of such piles is limited to dry beds and in soils where there are no under-currents of water.

Casting Piles in Flowing Water.

Captain M. R. Jefferis of the Royal Engineers has developed a system of driving a steel shell into the ground and driving the ordinary pile through this shell. This shell, called the casing, inside which the top of the pile is cast, protect the green concrete from erosion by water. Thus piles can be cast in runnning water and as the shell used has to be of a larger diameter than the tube, the top portion of the pile is cast thicker than the bottom thus giving it additional strength, where it is wanted the most.

To drive the shell into the ground, use is made of a circular shoe, called a ring-shoe, and a driving ring (both shown in Fig. 8). The ring shoe, as its name implies, is a circular cast iron ring the internal diameter of which is just larger than the ordinary shoe which passes through it. The slope of the outer face of the ring shoe is such that when the central shoe is placed in it, this face is in continuation of the outer face of the smaller inner shoe. This ensures a minimum resistance to driving. Into this ring shoe, when casting, a $\frac{1}{8}''$ thick steel sheet is burnt (shown as B in Fig. 8) and the top casing is rivetted on to this.

To drive the casing into the ground the smaller shoe and the casing, to which the ring shoe is rivetted, are assembled. The driving ring is lowered on to the casing and made to sit on the shoe. The tube is then brought to bear on the driving ring. A circular wooden frame, shown as W. F. in Fig. 2 is then inserted between the tube and the casing to ensure axial drive of the two parts. The whole assemblage is driven down with half strokes as the heavy strokes tend to separate the ring shoe from the casing. As soon as the final depth is reached, the driving ring is removed and the tube is brought back again to drive the lower shoe. A hole is thus made into the ground of a larger diameter on top and of a smaller diameter at the bottom. The pile is cast in the same manner as an ordinary pile described above. Into the top casing the concrete is rammed by hand if dry and if the casing is full of water the concrete is deposited at the bottom through the tube, when the water simply flows over as the concrete rises into the casing.

The Bhimber Bridge.

The paper is primarily written for bringing into prominence the various difficulties encountered in the casting of piles in the clay sub-soil of the bed of the Bhimber Nallah and the various factors which govern the load-carrying capacity of the piles. A brief description of the design of the Bhimber Bridge will not, therefore, be devoid of interest and is given below.

General Design.—Plate No. I shows the design of the Bridge in general. It consists of 40 spans, built in units of eight from expansion joint to expansion joint, varying in length from 16 feet to 23 feet. The variation in spans was necessitated by the fact that the decking slab (no beams being used) was not continuous towards the expansion joints. To keep the depth of the R. C. decking slab uniform, a reduction in span

on either end was necessary. Every eighth pier was made thick, and was built in brick masonry. The decking slab was anchored to this pier which was located midway between the expansion joints. This ensured the slab expanding from the thick piers outwards in both directions. The horizontal expansion force acting on the outer pier top was thus reduced to half of what it otherwise would have been if one of the two expansion joints did not function. This feature is particularly important in view of the fact that thin piers built on single-row pile-bents (see Figs. 10 and 11) possess no lateral stability particularly in clay soils where the top of the piles cannot be restrained from deflection towards the sides.

Piers and their Foundations.

The thin piers consisted of three reinforced concrete columns with a brick-in-cement filling between (*vide* Figs. 10, 11 and 12) and were founded on three 'Vibro' piles cast in one row, 21″ thick on top and 16″ thick at the bottom. Every thick pier was founded on six 16″ 'Vibro' piles, which were driven in two rows, at a batter of 1 in 15 (Fig. 13). The bed of the river consisted of sand for the top 7′ and plastic clay underneath; and piles were driven to a depth of 22′, *i.e.*, about 7′ in sand and 15′ in clay. A heavy reinforced concrete beam, connecting the top heads of the piles completed the foundation work.

Abutments.—The stability of piles in clay soils of uncertain moisture contents, like the one met with in the bed of the Bhimber Nullah, to inclined thrusts transmitted by the abutments being considered doubtful, well foundations were adopted for them. This chanced to be a happy choice as the piles supporting the piers soon settled under the load necessitating subsequent underpinning. Had the abutments been similarly founded, the trouble would have been enormous. The abutments were also built in brick masonry.

The Decking.

As shown in Plate II the superstructure consisted of an 18″ thick reinforced Concrete Slab, cast monolithic with the piers and designed to carry 10 British Standard units of loading, with a 50 per cent. impact allowance. The reinforcement of the decking is shown in Plate II and was taken from the full Bending Moment Curves, cutting each bar 40 diameters beyond the point, where it was not needed. The carriageway is 18′ clear between two footpaths, 2′ wide, on either side. The wearing coat consisted of 3″ of Mexphalte-grouted limestone with a sealing coat of Spramex.

Expansion Joints.

The expansion joints provided are a particular feature of the Bridge. The design has been shown in Plate III. As will be seen the decking slab is supported on three points. As the slab expands and contracts, the movement is taken up by the Channel Sections. The horizontal thrust transmitted by the expansion force to the top of the pier, which is

affected the most, is thus eliminated. The design for the expansion joint was supplied by Captain M. R. Jefferis, R.E.; and the manufacture was undertaken by the M. E. S. Workshop at Peshawar.

Execution. Pile driving for foundations of piers.

Work was commenced in March, 1931, from the left bank. The bed was nearly dry and the foundation work was proposed to be finished before the flood season of 1931.

A test pile was driven to a depth of 25′; and the set obtained was $\frac{1}{2}$″ for a $1\frac{1}{2}$ ton hammer, dropping 4′—6″. This gave a load-carrying capacity of 56 tons for the piles from tables given by the 'Vibro' Piling Company, giving a factor of safety of two which was not considered ample. The piles were bulged at the bottom to give a bigger bearing area.

To obtain a bulged pile for the bottom three feet, the lower two feet length of the pile was cast as usual, and the tube, together with the concrete, hammered down one foot. The next two-feet length was then cast, and the pile hammered down one foot again, and similarly for the third time. The level of the concrete inside the tube, was continuously meaured by dropping a plumb bob from the top and the extra concrete going into the bulged portion was measured. Usually 2 to 3 cubic feet of extra concrete was utilized. This was supposed to give extra stability by increasing the bearing area of the bottom portion of the pile. All piles were bulged at the bottom in this way.

One great difficulty felt in this process was that, as the tube was extracted, the plastic clay of the bed stuck to the tube and came out with it. With continuous vibration, a portion of it dropped into the casing, and when bulging the bottom three feet (which meant hammering the tube down), a quantity collected near the ring shoe at the bottom of the casing. Before casting the pile on the top portion, however, every effort was made to scrape and remove the clay, in spite of which, as transpired later on when the piles were stripped for underpinning the structure, they were never found to be perfect near the junction. Very often the section of the pile was only three-quarters of what it was supposed to be, the remaining portion being occupied by clay which could not be removed. Fortunately, as described later on, these deformities proved of great help in bonding the footings to piles when underpinning the structure.

The author is definitely of opinion that it is very difficult to avoid clay getting into the junctions of the thick and the thin piles. When conditions similar to the Bhimber Bridge obtain and clay sticks to the piling tube, it is preferable to cast piles of one diameter throughout and lower the ground transom, connecting the piles, to a point below the scour depth, if possible.

As the work proceeded, it was noticed that the casing could be driven more easily than described earlier. Instead of the expensive cast iron ring shoes described previously, a mild steel L iron bottom shoe was

designed, as shown in Fig. 10, and worked very satisfactorily. A saving of about Rs. 5,000 on the whole job was effected by its use. Such a shoe can, however, only be used in clay soils and sand and will be of no use in shingle and boulder beds of rivers.

The piling work was completed without any difficulty working single-shift for the first two months, when an accident occurred and the piling plant turned over while driving the piles. The bottom roller of the plant rested on wooden sleepers, which spanned the holes dug just below the piling tube down to the level of the top of the finished pile. The bed being sandy, a simultaneous slip occurred in two of such holes and the span of the wooden beam, supporting the piling plant, increased from four to elven feet, snapping it and tilting the plant. The accident is mentioned, just to bring into prominence the importance of taking all necessary precautions for properly supporting the plant. The damage done to various parts necessitated stopping the work for two weeks, after which two shifts of labour were employed and the work completed before the monsoon.

Shuttering for Decking.

For erecting the shuttering for decking, holes were left in the piers. The centering, consisting of 2″ planks, was supported on wooden sleepers laid on end on R. S. beams spanning between the piers and supported on R. S. joists, laid parallel to the same, resting on 12″ × 12″ wooden logs passing through the holes in them. To allow for the eccentric loading and lifting up of the timber pieces, when one span was loaded, the long beams were strutted from the transom offset (see Plate IV). The side curves, were made of M. S. Sheets, bent to the proper curvature. Enough shuttering was made to cover 10 spans, so that work was more or less continuous, two spans being poured each day. Fig. 14 shows the concreting of the decking being poured during a heavy flood.

Settlement in the Foundations.

The first flood came on 13th August, 1931, by which time the decking of eight spans had been completed. As the floods could conveniently pass below the temporary centering, the work went on unhampered (see Fig. 14).

Immediately after the 2nd set was poured it was noticed that the centering had got wavy and the shuttering for the footpath, which was erected dead level had shown as much as $\frac{3}{4}$″ settlement in a few spans. It was at first thought to be the yield of the shuttering due to bending of its various members under the weight of the slab; but on closer examination, even the first eight spans, which had been erected very successfully and the footpaths made dead level, showed signs of sagging and cracks appeared in the footpaths at right angles to the spans of the bridge practically continuous from one end to another.

Sand was removed from above the next eight spans, and similar cracks were observed in that slab also. Apparently the foundations had

settled. To make sure, levels were taken, both of the top of the wheel-guard and of the transoms of piers, and a settlement of about $\frac{3}{4}''$ noticed. It was at first ascribed to the elastic yield of the piles and the soil. Careful record was, however, kept of all the levels ; and it was found that the settlement was progressive. Apparently the piles were overloaded and could not carry the weight, imposed on them, without settlement.

Further work was stopped, and the levels were continuously watched. Finally a settlement of $3\frac{1}{2}''$ was registered in some of the piers ; but luckily the decking slab, already laid, did not show any signs of cracking. It was, then, decided to underpin the piers before pouring the decking of the remaining spans. The problem of underpinning presented many complications. There was no chance of adding a few more piles under the piers, which would perhaps have been possible by using piles cast by compressed air piling system.

The alternatives were to sink two wells between the three piles, or else to reduce the load on the piles somehow by transmitting a portion of it directly to the clay which was located below the depth of scour. Both methods were extremely difficult, but the former more than the latter, which was, therefore, finally adopted. It was accordingly decided to add on a reinforced concrete circular footing, 6′ in diameter, at the junction of the thick and thin portions of each pile. The detail of this footing and its reinforcement is given in Plate V.

To put this footing in position, circular steel casings, 7′ diameter and 3′ high, were made in two halves, which could be bolted together with the help of angle irons fixed to them. Having made an excavation in the ground, under the transom, the two halves were brought to form a concentric ring round the pile and were bolted together. The casing was then gradually lowered by being sunk like a well, keeping the bed inside dry by pumping. As soon as the upper end of the casing was lowered up to the excavation level, another similar casing in two halves was added on at the top and bolted to the bottom one. Both the casings were then sunk together, and on striking the excavation level a third piece was put on in the same way as before, and the whole sunk further. Loading of the casing became necessary for sinking after it went a few feet into the ground and friction on the sides developed. The work was executed by petty contract (dealing with the labour direct) at a small rate of Rs. 5 per foot for wet sinking, the pumping charges being borne by the Department.

As soon as the proper depth was reached, which was mostly dependent upon striking good clay on which to found the footings, sinking was stopped. One of the greatest troubles experienced was that, as soon as casings struck the clay and were left overnight, about two feet of the top clay turned into slush. This had to be removed next morning before the work of sinking of the casing could proceed further. The problem became so acute that it was at one time considered that if the soil got so saturated under a small upward flow of water, its stability under the full load was extremely doubtful, and the additional strength

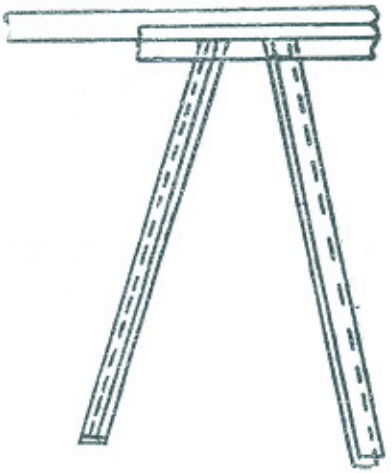
o be gained by underpinning would be negligible. Reliance was, however, put on the fact that so long as the clay did not come in contact with ir, slush did not form; but the point requires further research and nvestigation. Footings were founded on this clay, always towards the vening so as not to allow the slush to form.

To further reduce the weight on the piles and footings, that around he central pile was substituted by a 7′ diameter masonry one, sunk like n ordinary well. The offset of the pile was buried in the concrete of he hearting of this well; and two columns of masonry (1′ 6″ × 1′ 6″) rected on the steining of the well so as to come up to the transom and ive an intermediate support between the piles (Plate VI).

As soon as four piers were thus underpinned, the shuttering was rected, and each pier loaded with sand to twice the load that the structure was expected to bear. It was occasionally discovered that settlement, arying from $\frac{1}{4}$″ to $\frac{3}{4}$″ depending upon the compressibility of the soil, lid occur. This was however taken as inevitable and no danger was nticipated, which has since proved to be true by the fact that after the ompletion of the bridge, the floods of 1932 rose to the level of the highest lood level, and no settlement took place. The whole work of underpinning took about six months, or the whole of the cold weather.

Settlement in Thick Piers.

Luckily the thick piers had not settled, although the load per pile was the same as for the thin ones. The author is of opinion that the inclined piles driven in pairs as per sketch on the right, offer a greater resistance to settlement than vertical piles. Before the piers settle, no movement of the inclined piles in the direction of their length can take place without the pile racking. If they may be assumed to settle so as to occupy a position as per dotted lines, the bearing of the whole of the pile on its inner surface adds to its load-carrying capacity. The author is definitely of opinion that, in the case of clay soils, where the piles possess a low point resistance at the bottom, inclined piles should be used, since they give additional bearing area on the underside of their whole length.



From the above it has been seen, that the piles in the foundations of the Bhimber Bridge had settled under half the load which they were expected to carry, as calculated by the Hiley formula (explained below). The load-carrying capacity of a pile is computed from the set (or penetration) of the pile under the impact of the hammer, by various formulae. In certain soils this capacity increases with a period of rest, and in others it decreases. It is, therefore, absolutely essential that we should differentiate between Driving Resistance (D. R.) and ultimate Bearing

Resistance (B. R.) to dead load brought to bear on the pile after the ground all round has come back to a state of equlibirium. The former is the force brought into play by the ground in resisting the driving of the piles by successive hammer blows ; and the latter is the resistance offered to a pile, which keeps it back from penetrating into the soil under static conditions.

It is also clear that the values for D. R. and B. R. are not equal. Again, the Driving Resistance (as calculated by the formula) and also Bearing Resistance both depend upon two factors ; *viz.* :—

(1) The friction of the soil on the side, and

(2) The resistance to penetration of the point at the bottom, offered by the ground area equal to the horizontal projection of the cross section of the pile.

When the pile is being driven under an impact of the hammer, the resistance overcome consists mostly of point resistance, the side friction being very small.

Tables supplied by the 'Vibro' Piling Company, on which the calculations for the load carrying capacity for the piles of the Bhimber Bridge were made, are based on the semi-elastic impact theory, and allowance is made for the side friction separately. As a matter of interest this theoretical formula has been given below, just to mention the improvement on the old pile driving formula, the derivation of which is also given :—

(1) **Old Engineering News Formula.**

If S is the penetration (average for the last 10 blows) in inches
H is the free height of fall, and 6 (six) the factor of safety.

Equating the energy of the hammer to the work done in driving the piles s″ down, we have

$$12\ M\ H = (S + 1'') \times \text{Resisting force overcome.}$$

The safe load-carrying capacity of the pile is, thus,

$$= \frac{12\ M\ H}{6\ (S+1)} = \frac{2\ M\ H}{S+1}$$

The value of 1″ has been based on actual bearing values after a period of rest.

It will be seen that, in working out the above formula, the following factors have not been taken into account :—

(1) Energy expended in compressing the cushion or pile cap.
(2) ,, ,, ,, ,, the pile.
(3) ,, ,, ,, ,, the quake of the ground.
(4) Efficiency of the hammer blow.
(5) Loss of energy due to the two surfaces coming into contact depending on the co-efficient of restitution of the two materials.

The Hiley formula takes into account some of these factors and has been derived as follows* :—

To find out the efficiency of the blow, let us take

V to be the velocity of the hammer before impact.
U ,, ,, ,, ,, ,, ,, after ,,
v ,, ,, ,, ,, ,, pile ,, ,,
M is the Mass of the Hammer.
P ,, ,, ,, ,, ,, Pile.
g ,, ,, gravitational constant.
E ,, ,, co-efficient of restitution.

Equating moments before and after impact, we have

$$M V = M v + P u \qquad \text{(i)}$$

The relation between velocities before and after impact is—

$$v - u = -E V \qquad \text{(ii)}.$$

Eliminating v between these two equations, we have

$$u = \frac{M}{M+P}(1+E)\ V \qquad \text{(iii)}.$$

and eliminating u between the same two equations,

$$v = \frac{(M-EP).\ V}{M+P}$$

The hammer will be able to maintain a downward velocity only if M is greater than EP and will rebound, if it is less than EP.

The sum of the energies existing in the hammer and the pile $= \frac{1}{2g.} M\left(\frac{M-EP}{M+P} \cdot V\right)^2 + \frac{1}{2g} \cdot \frac{P}{1}\left(\frac{M(1+E)}{M+P} \cdot V\right)^2$

$$= \frac{MV^2}{2g} \cdot \frac{1}{(M+P)^2}\left\{M^2 + PM + PE^2(M+P)\right\}$$

$$= \frac{MV^2}{2g}\left\{\frac{M+PE^2}{M+P}\right\}$$

Since the initial energy of the hammer was $\frac{MV^2}{2g}$ and $\frac{V^2}{2g} = h$, (the height of fall) :

and if $\frac{M+PE^2}{M+P}$ be termed as e, indicating the efficiency of the blow, the **energy** expended $= eMh$

It will be seen that when the impact is between elastic bodies $E = 1$ and $e = 1$; and between completely inelastic bodies $E = 0$ and efficiency of blow $= \frac{M}{M+P}$

* (Structural Engineer (England) July, 1933.)

Out of the various factors enunciated previously, Hiley accounts for the following :—

(1) **Temporary elastic compression (cp.) of the pile,** which is a function of the resistance offered to the pile (cp.). The energy lost in this item in compressing a pile under a resisting force R (varying from zero at the beginning of the stroke to R at the end)$=\frac{1}{2}$ R $\times$ cp. and

cp.$=\frac{Rl}{EA,}$ when R is the force producing compression—

l is the length of the pile.
E is the co-efficient of elasticity of the material of the pile.

(2) **Temporary extraneous compression of the ground** manifested by the quake of the ground during the downward movement of the pile at each blow, (cq.) which may average 1″ in firm gravel and 2″ in fine clay.

(3) **The compression of the pile helmet on top** compressing the cushion, which may be called Cc. So that the actual Set (s) observed in the gauge, is actually s—(cp+cq+Cc).

Calling the three items within brackets as C, (an equivalent compression) the driving resistance formula may be worked out as below :—

Energy expended, as worked out above,

$$=\frac{MV^2\ (M+PE^2)}{2g(M+P)}=Mhe$$

and the work done in driving the piles and compressing the various parts enunciated above

$$=RS+R\frac{c}{2}$$

R is ground resistance
S is the penetration, or distance travelled by the pile.

$$=R\left(s+\frac{c}{2}\right)$$

Equating the two, we have

$$R=-\frac{M\,h}{\left(S+\frac{c}{2}\right)}\times\left\{\frac{M+P\,E^2}{M+P}\right\}=\frac{e\,Mh}{\left(S+\frac{c}{2}\right)}$$

and eh may be termed the equivalent effective height, which is 92 per cent. of the actual drop.

It will be seen that the value of c, (equivalent compression) depends upon R (the driving resistance), and, therefore, a value of C is assumed and R found, till by trial land error the results are consistent. The value of e has been found by experiments for various conditions.

It will be seen that Hiley's formula takes into account various factors, but still ignores some of them. Till such time as a more accurate and reliable formula is arrived at by further research work on the development of the piling formula, this has been accepted as representing conditions nearest those actually met with ; but there are many limitations to

s formula, as has been demonstrated from the results of piling at the imber Bridge, in which case the piles, although designed to carry the l load according to the Hiley formula actually failed. That the piling mula has given accurate and satisfactory results in a number of bridges the Frontier Province and at the Wah Bridge where the piles were ven in boulder bedded rivers, is an established fact. At the Bhimber dge in a clay soil it did not. In other words, there was no difference ween the Dynamic Resistance and Ultimate Resistance, in one case, ile there was a considerable difference in the other.

Apparently then the differential behaviour is due to the two kinds of porting strata met with. The science of soil mechanics is still in infancy. The effect of (*a*) mineral composition, (*b*) volume of voids, grain size, by mechanical analysis, (*d*) water and moisture content and the percentage of colloidal material present on bearing Value of Soils foundations, as reported by the special committee of U.S.A., has ealed results which are not conclusive. Till such time as soil me- nic establishes some definite relation between the various soils and ir bearing power, soils have been classified as follows for pile driving poses * :—

(1) Sand, Gravel and Permeable artificial fills.

(2) Fine-grained silts and plastic clay materials.

the first case, the pile driving formulae give accurate results ; while he second case they do not, a fact which has been established from our erience in the two kinds of soil mentioned above. The values of D. R. . B. R. for the first kind of soil are identical and for the second they y be the same, or widely different.

planation of the Differential Behaviour of Piles.

It has been discussed above that, while driving a pile, the resistance red by the ground is far in excess of that offered by friction on the s, due to the fact that water squeezed out by the pile gets between the and the pile. In the case of ' Vibro ' Piles the bulge of the tube at bottom makes a hole larger than the tube, and the side friction is gether cut out. In clay soils this is particularly so. The work done lriving the piles into saturated clay, which results in forcing the water is so large that the values of D. R. are considerably increased, and ch fall considerably as soon as normal conditions are restored by rest. side friction almost invariably increases after a period of rest, and pensates partially for the loss occurring due to the reduction in point stance. In certain soils the losses and gains may thus balance each er ; but unfortunately in the case of Bhimber Bridge they did not.

v to differentiate between two kinds of soil.

The present method of classifying soils by sight and then estimat- their safe loading value from previous experience is bound to give

* Terzaghi—On Science of Foundations—Paper 1704, American Society of l Engineers.

wrong results. The best method of classifying soils in the case, where piles are to be driven, is to cast some test piles and load them ; or if the work is to be quickly done, they should be redriven next day after a rest of at least 24 hours. If the Driving Resistance, as calculated from the observations of the penetrations on the two days are the same, the soil can be classified as No. 1, otherwise No. 2. In the case of No. 2, the results should be verified by a test load on the pile ; and if the soil is dry, but liable to get wet after some time, both the conditions should be investigated by direct loading. If the above precautions are taken, pile foundations can be safely depended upon.

Individual Piles and Groups of Piles.

Usually there is an impression that ' n ' number of piles will take ' n ' times the load, which one pile can bear ; and that, with the compression of the soil all round in a group of piles, the bearing value of each will increase. In soils of class (1) piles compress the materials during driving ; but in the case of soils of class 2, pile driving is accompanied by a subsidence of the soil all round in some cases while in others it results in upheaval. In the former case, the bearing capacity of a pile is slightly increased (provided they are not less than 3 feet apart); while, in the latter, it is always uncertain. To be definite in clay soils it is, therefore, imperative that piles should be loaded with a test load and their bearing capacity found out. From the above it is clear that an individual pile in a group may have its bearing value increased or decreased by adjacent driving. Another fact is very definite that an individual pile distributes the load over a large area while in the case of a group the loaded area is not directly proportional to the number of piles.

Conclusion.

From the above description it is definitely established—

(1) That the existing driving formulae are only applicable to soils such as gravel, sand, etc.

(2) That in clay soils the bearing value of a pile after a period of rest can be considerably less than it was at the time of driving.

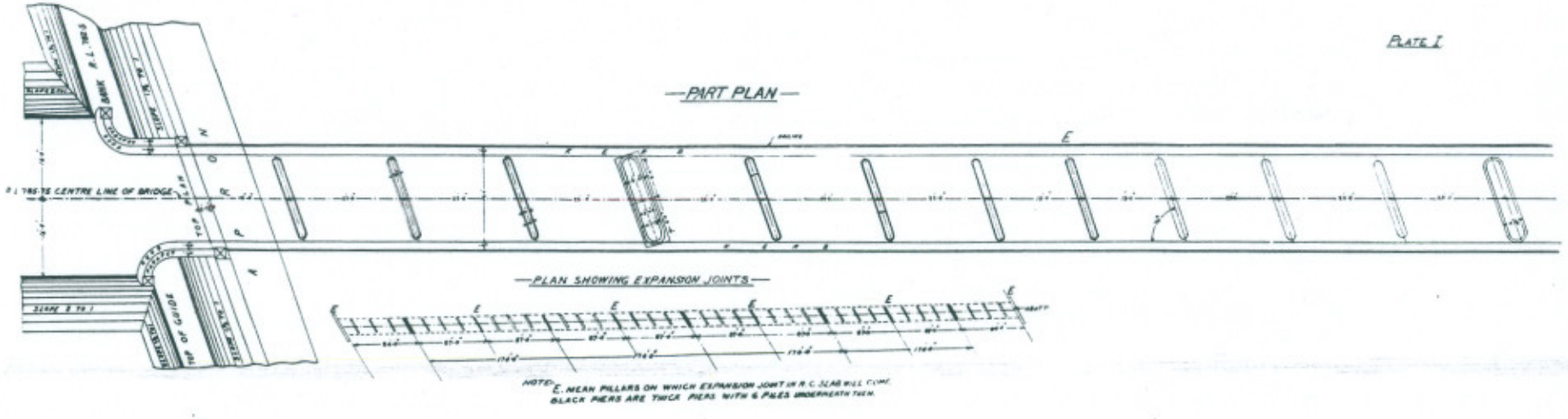
(3) Two diameter piles (*i.e.*, thick on top and thin at the bottom) should not be used in soils which stick to the tube. This leads to a defective pile at the junction.

(4) That inclined piles distribute the load by their underside acting as a bearing area, and consequently are very useful in clay soils, and should invariably be used in groups of two, as under the thick piers in the case of Bhimber Bridge.

(5) That a group of piles may take less load than calculated from the bearing capacity of one individual pile.

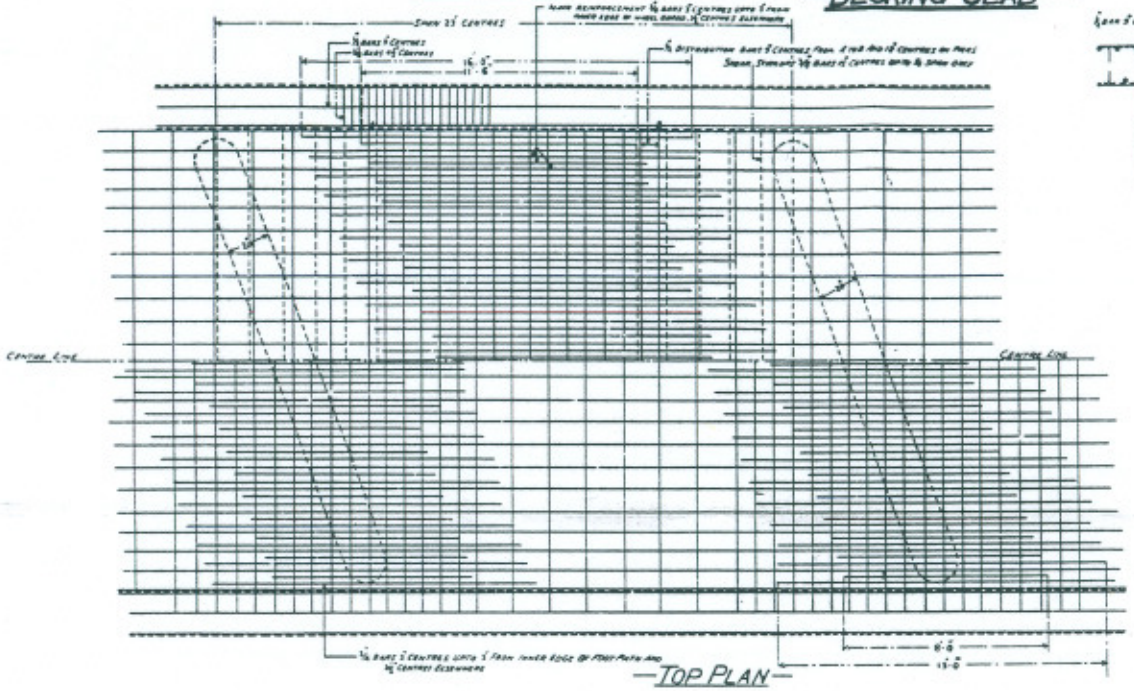
The above work was executed by the author as Assistant Engineer-in-charge of Construction under the guidance of R. B. Sant Ram, Superintending Engineer.

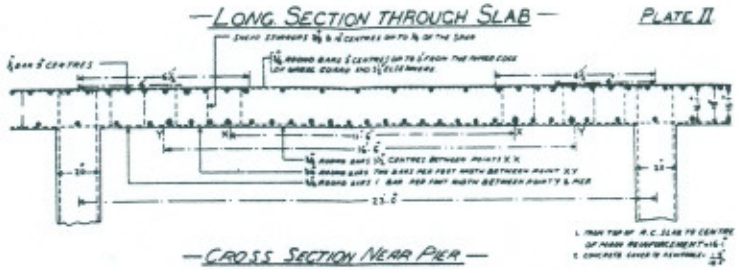
PLATE I

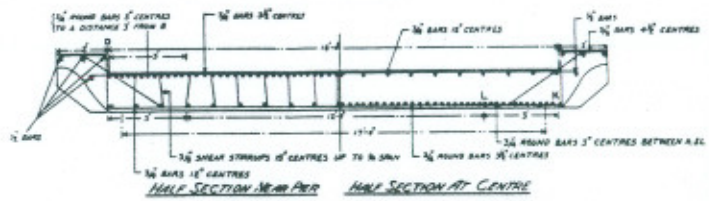

—PART PLAN—
SLOPE 2 TO 1
SLOPE 2 TO 1
—PLAN SHOWING EXPANSION JOINTS—
NOTE:- E. MEAN PILLARS ON WHICH EXPANSION JOINT IN R.C. SLAB WILL COME.
BLACK PIERS ARE THICK PIERS WITH 6 PILES UNDERNEATH THEM.

—DECKING SLAB—

PLATE II



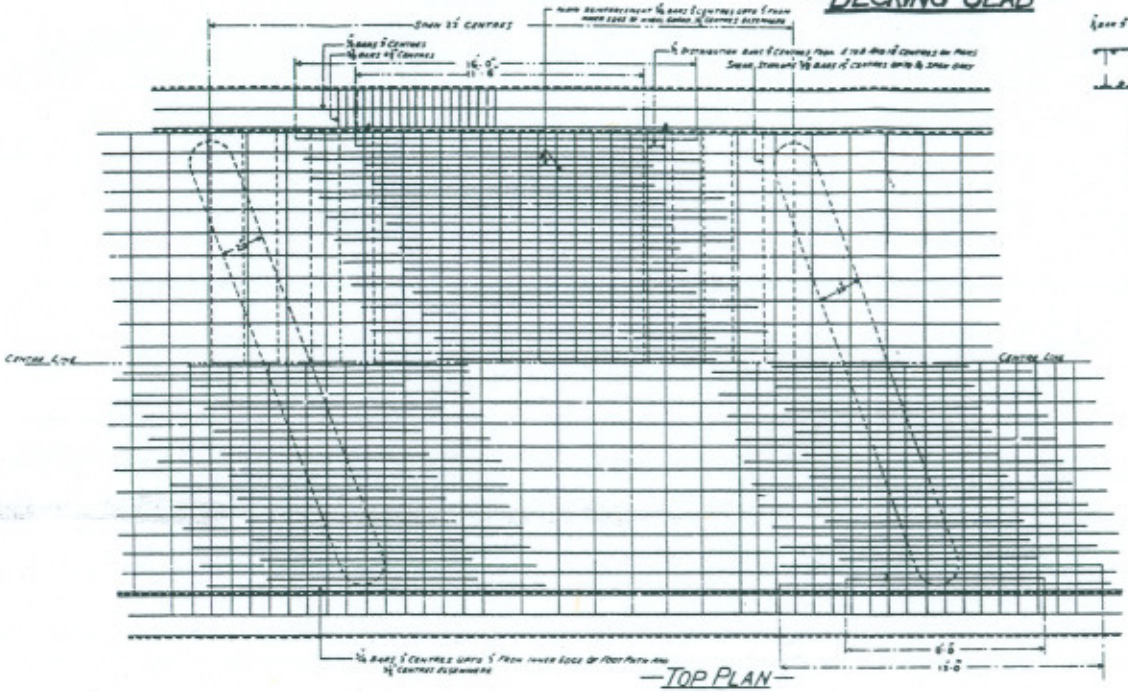


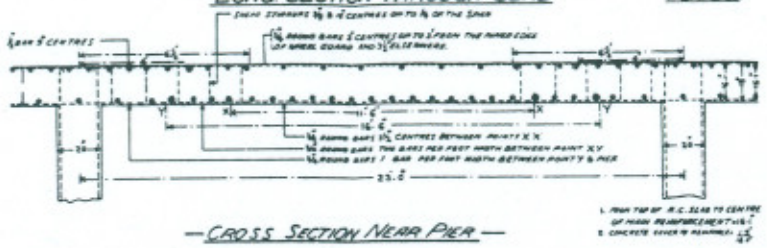


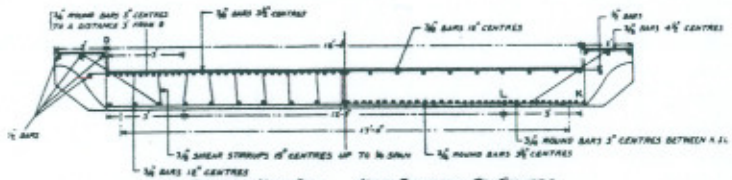


—DECKING SLAB—

PLATE II







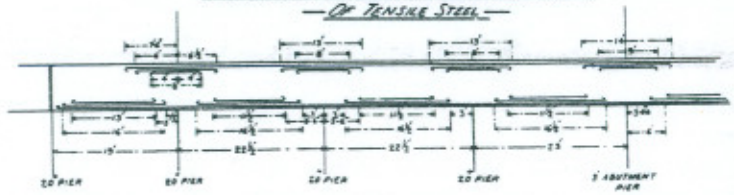
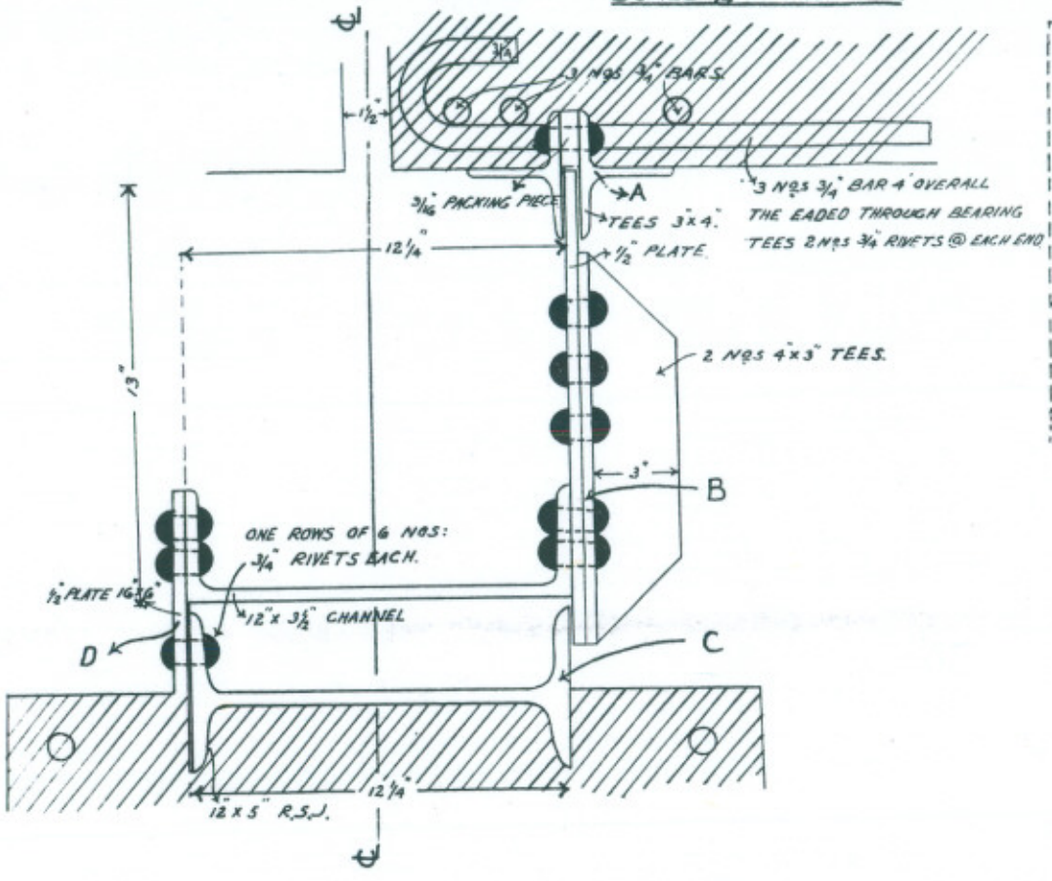
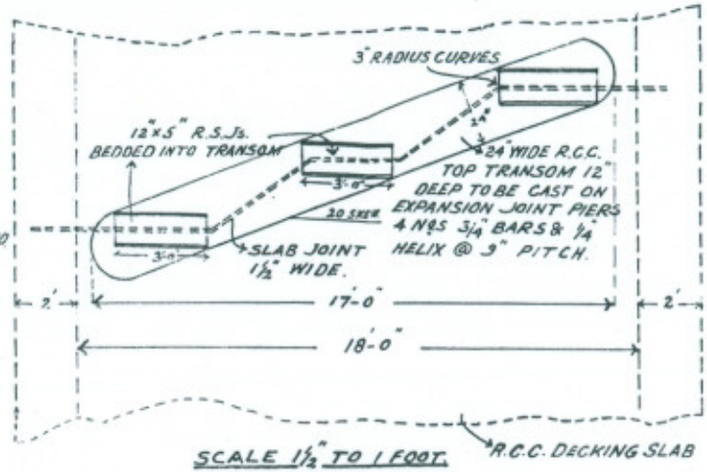


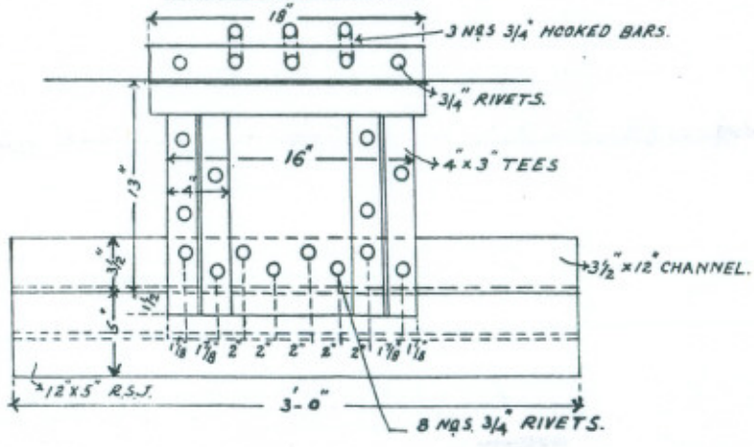
PLATE III.

SCALE ¼ OF FULL SIZE.





SCALE 1½" TO 1 FOOT.



FORM-WORK FOR R.C. DECKING

PLATE IV.

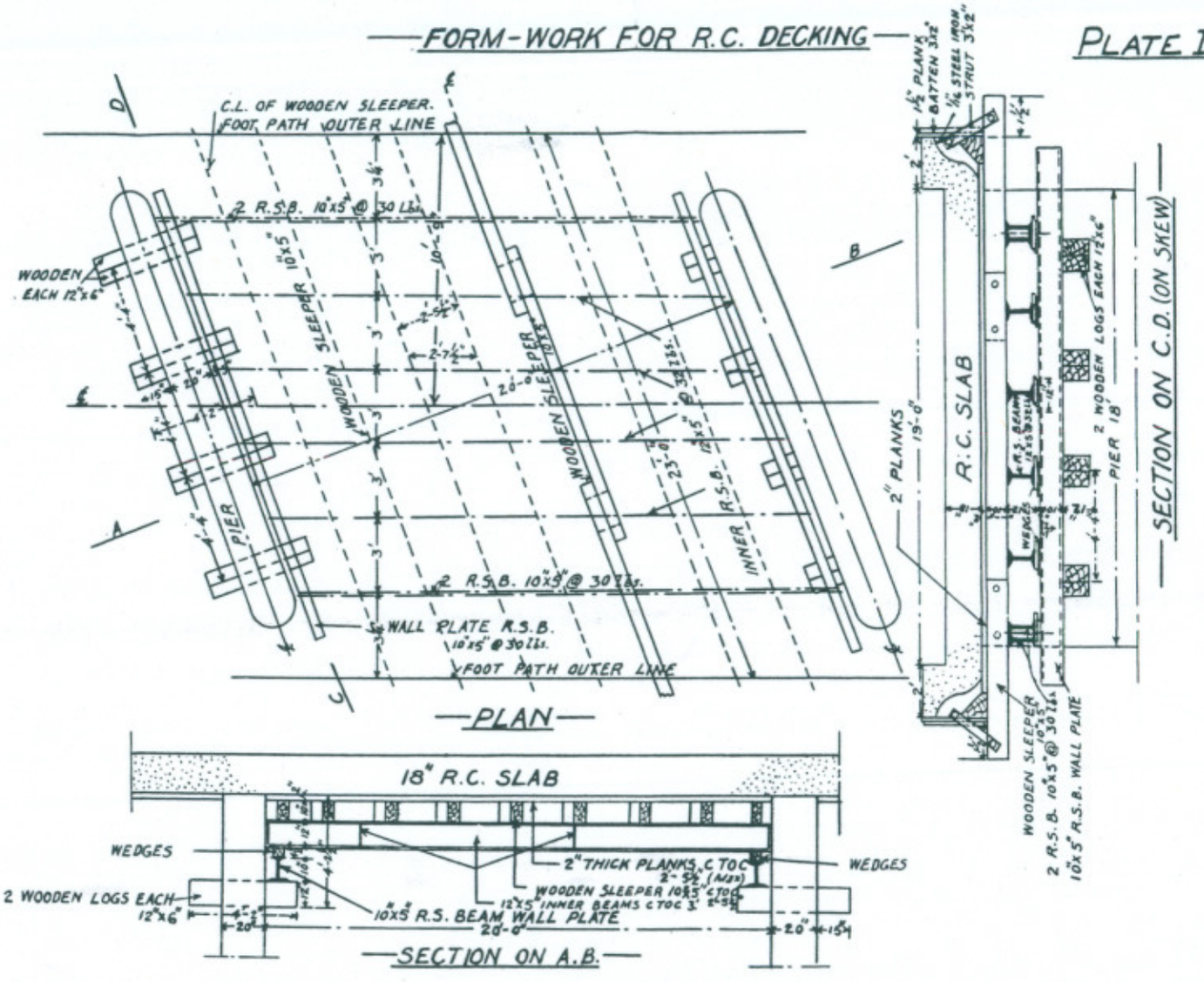
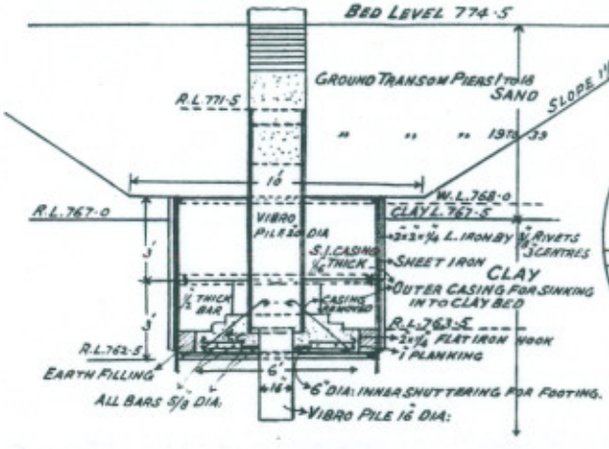
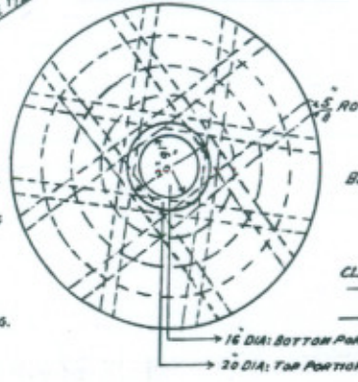


PLATE V.

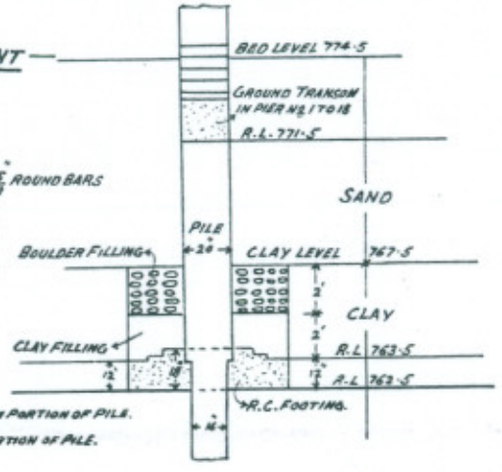
FOOTING FOR PILES

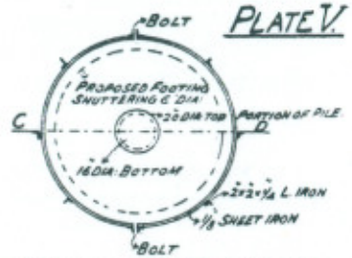


PLAN OF REINFORCEMENT



SECTION OF FOOTING





PLAN OF OUTER CASING

SECTION ON C.D.

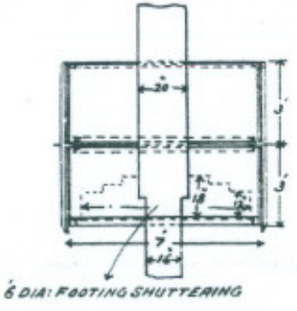
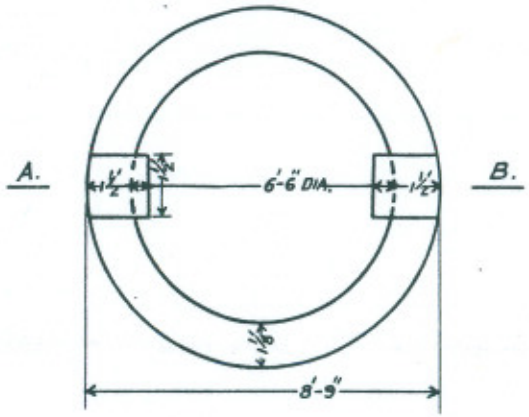


PLATE VI.

—THE WELL ROUND THE CENTRAL PILE OF EACH PIER.—

PLAN



SECTION ON A.B.

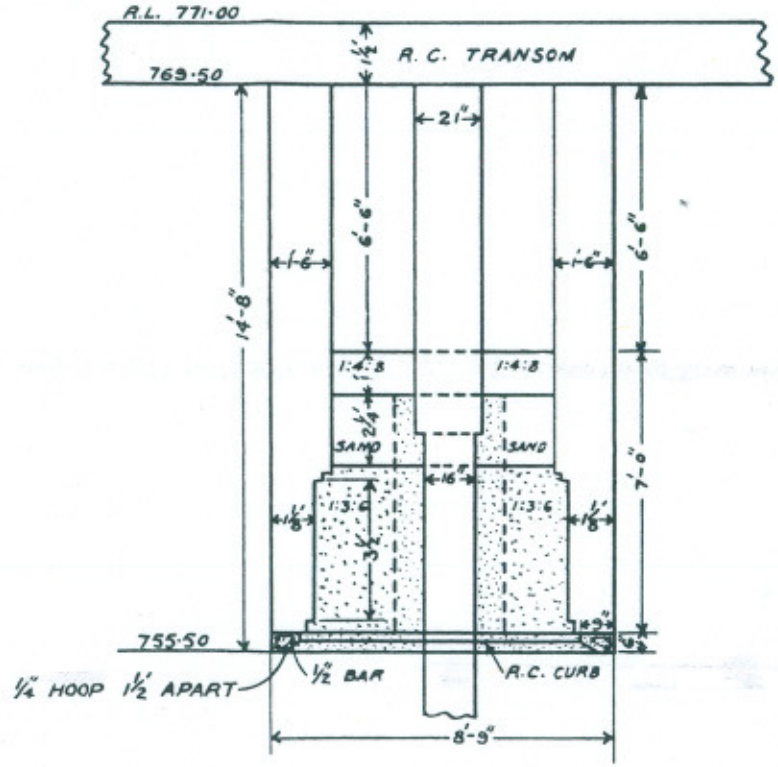
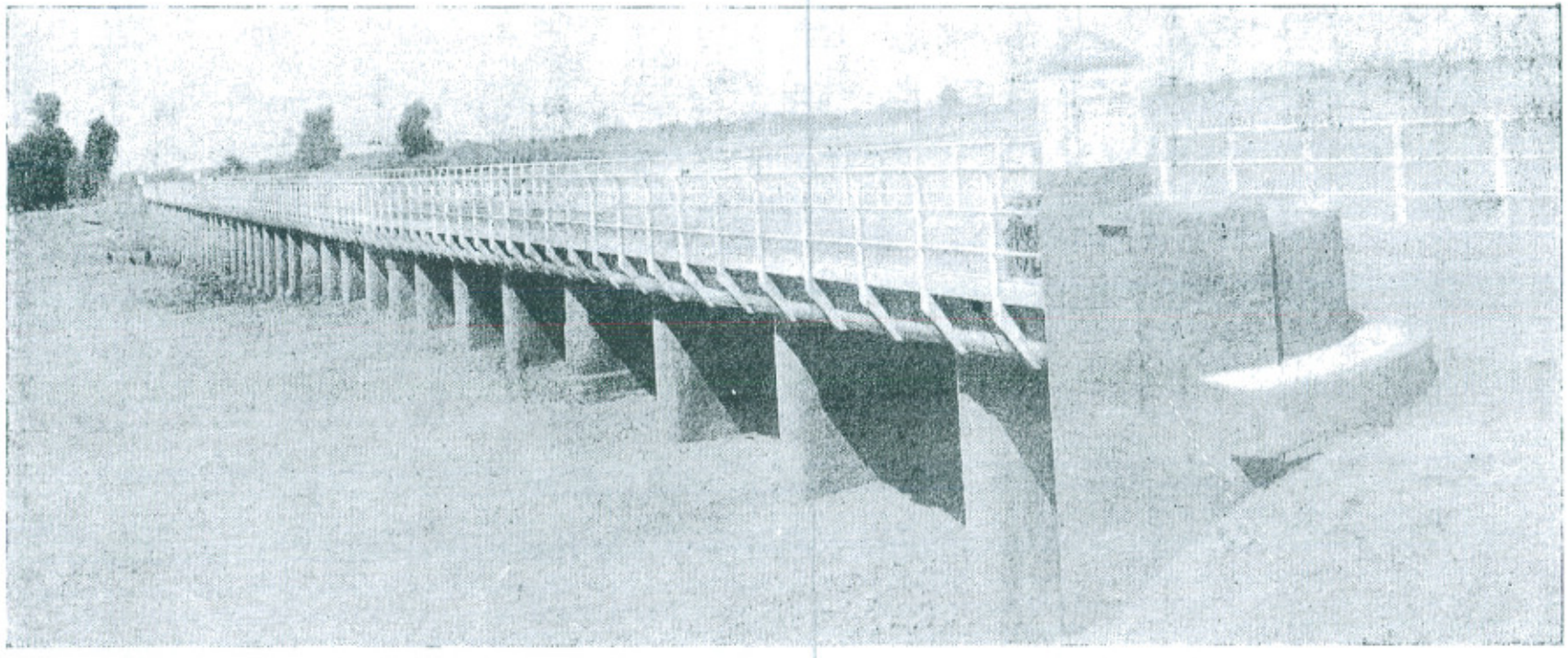
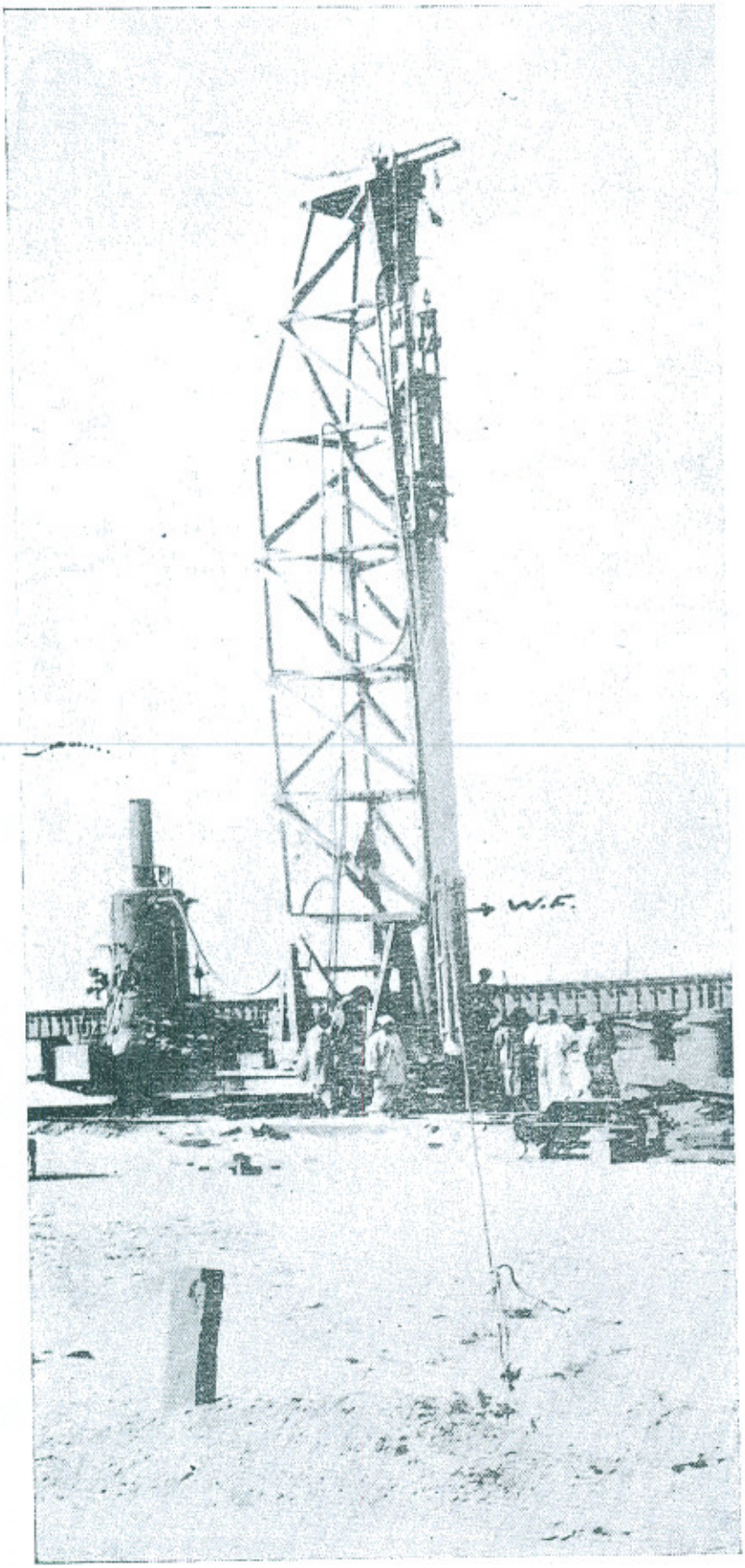


Fig. 1.



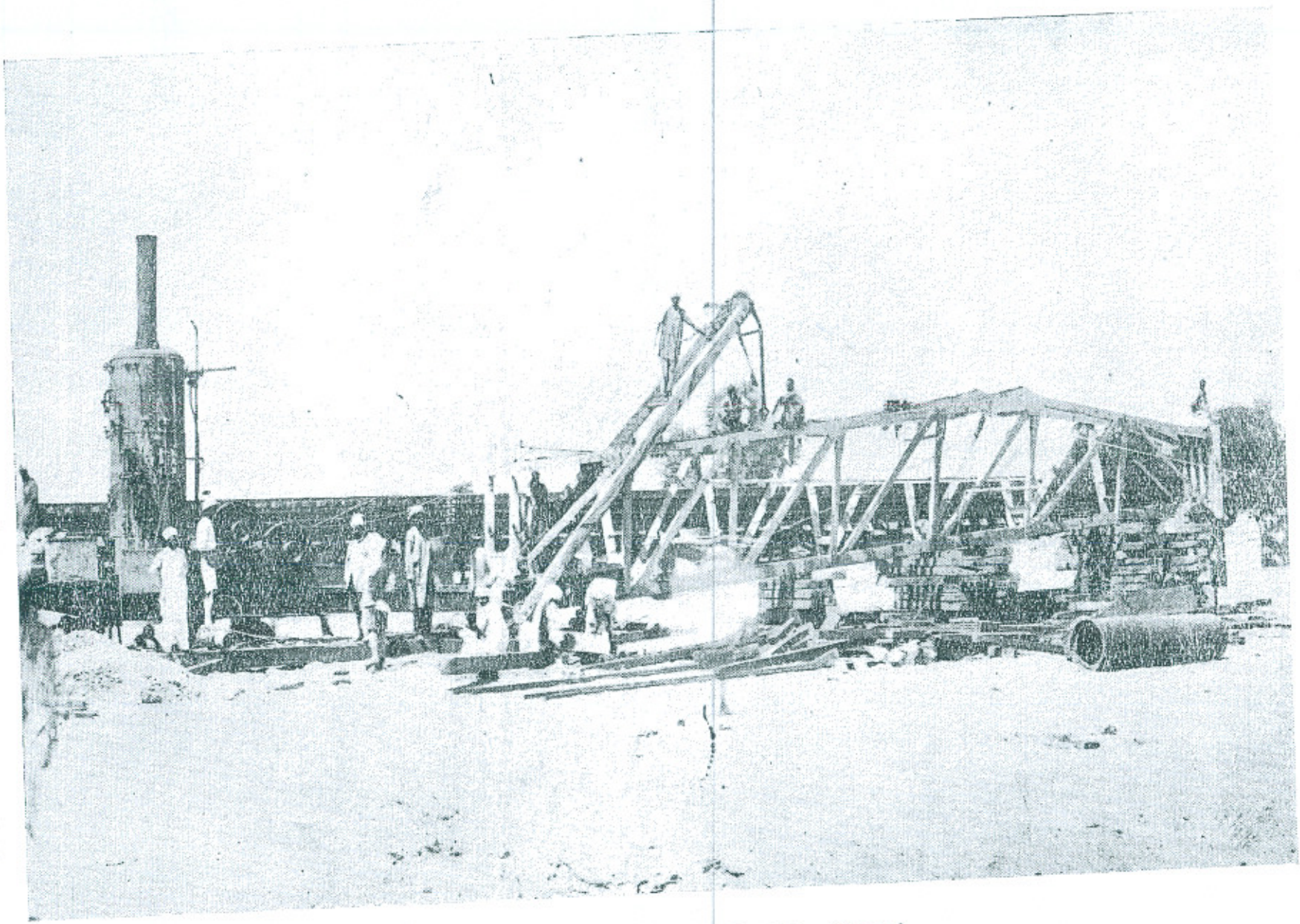
Bhimbar Bridge, Built 1932.

Fig. 2.



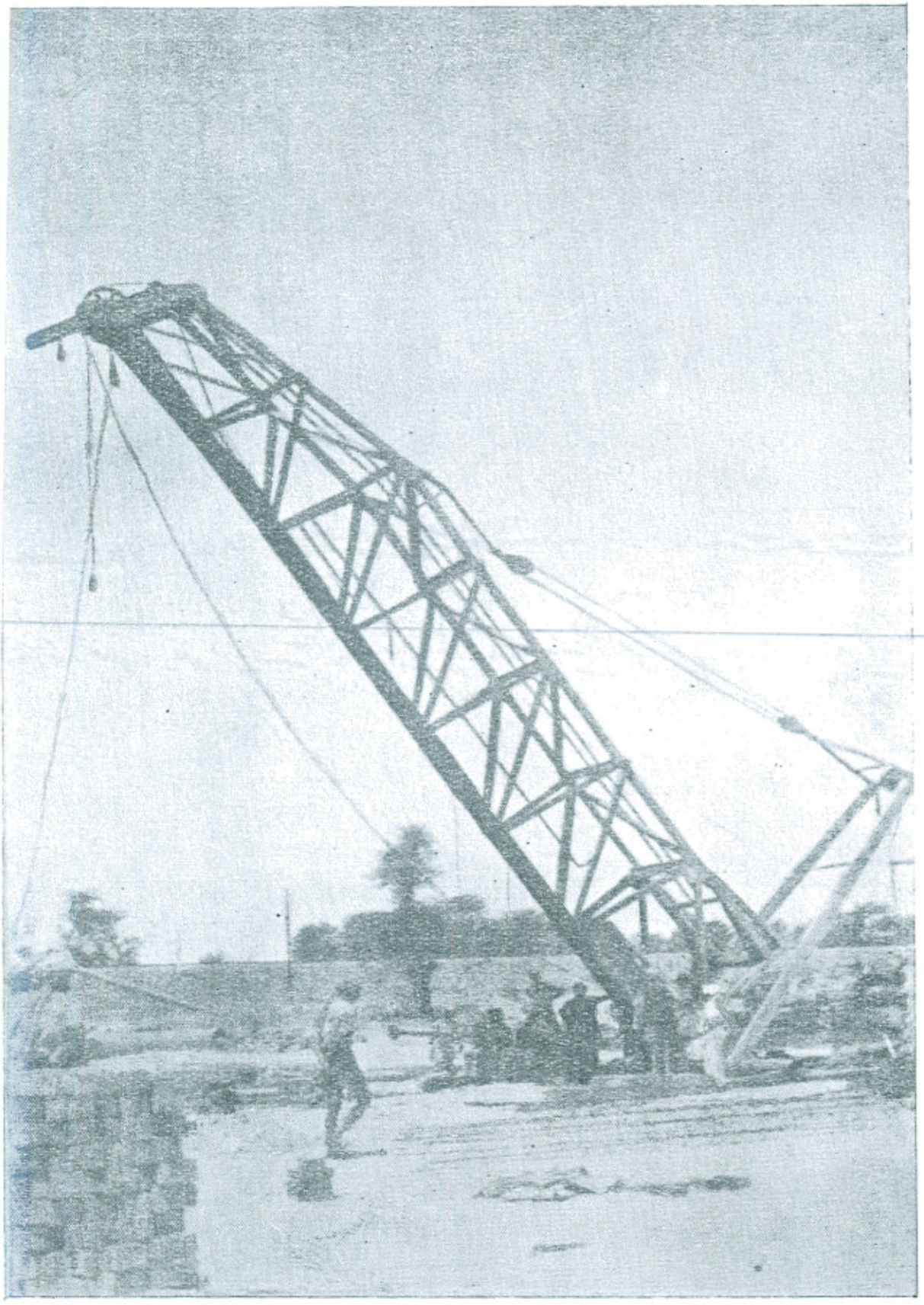
Vibro Piling Plant.

Fig. 3.



Jib of Vibro Piling Plant as Erected on Ground.

Fig. 4.



Vibro Piling Plant, Jib being Hoisted up.

Fig. 5.

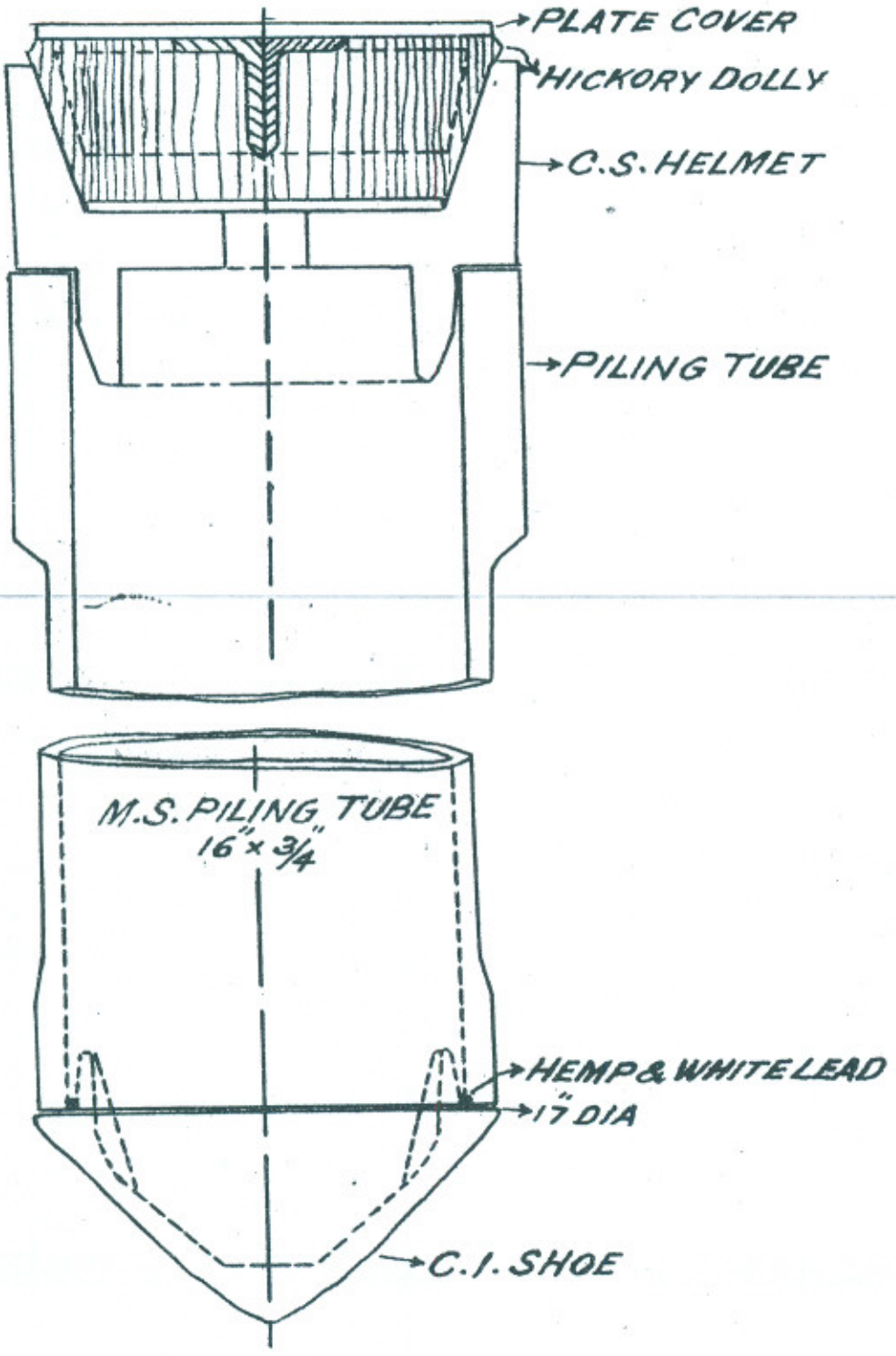
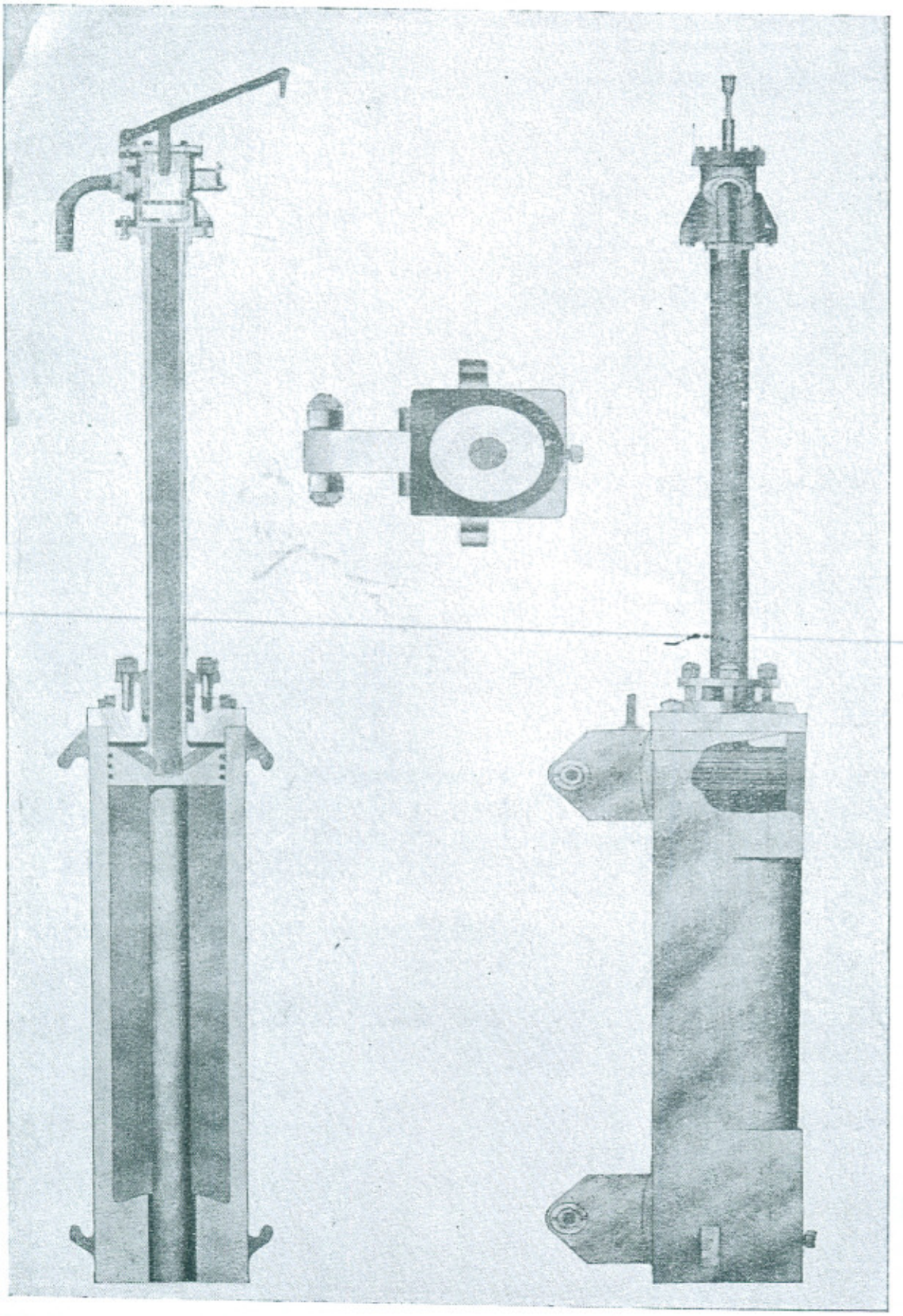

PLATE COVER
HICKORY DOLLY
C.S. HELMET
PILING TUBE
M.S. PILING TUBE
16" x 3/4"
HEMP & WHITE LEAD
17" DIA
C.I. SHOE

Fig. 6.



The B.S.P. Single Acting Hammer.

Fig. 7.

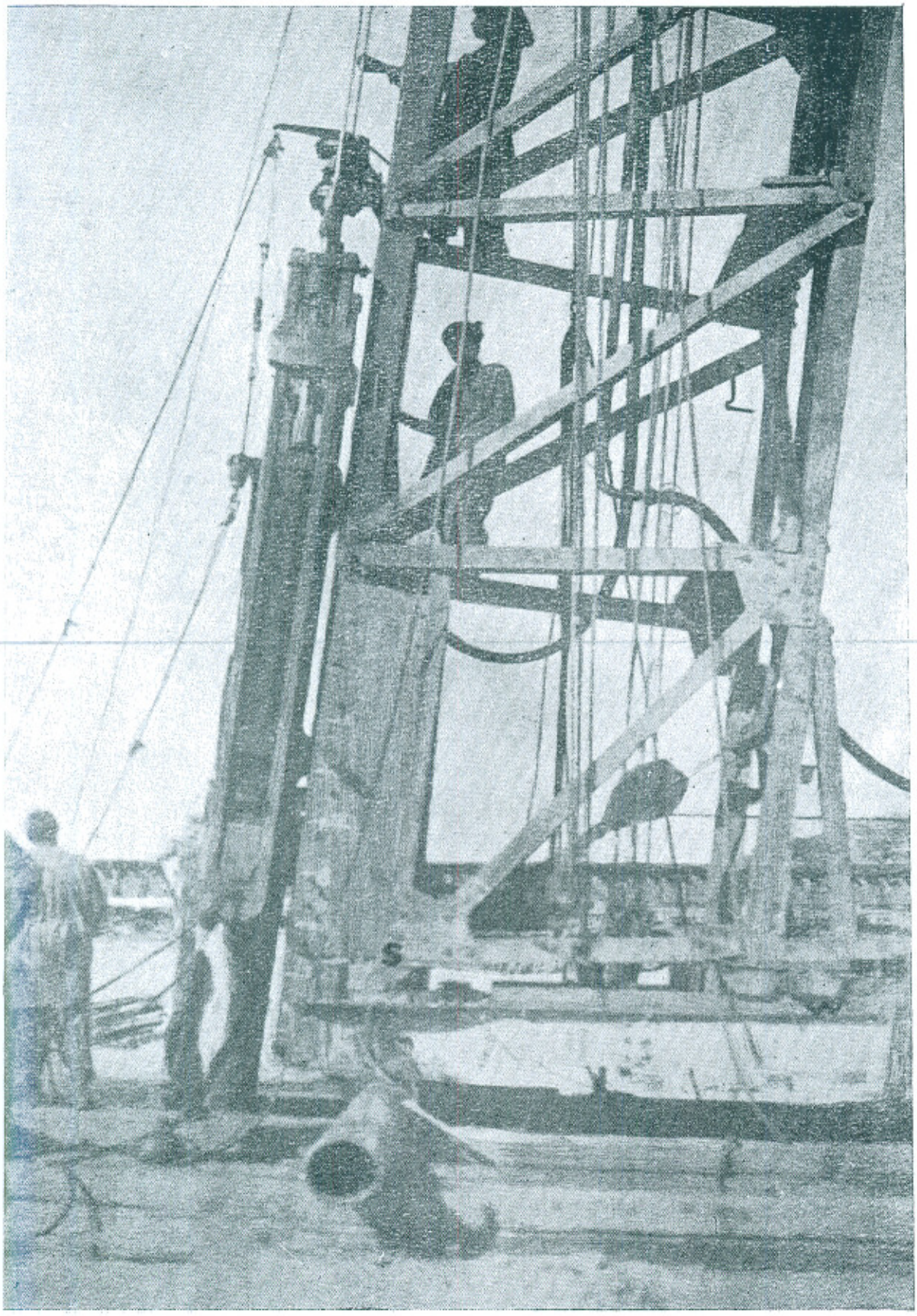
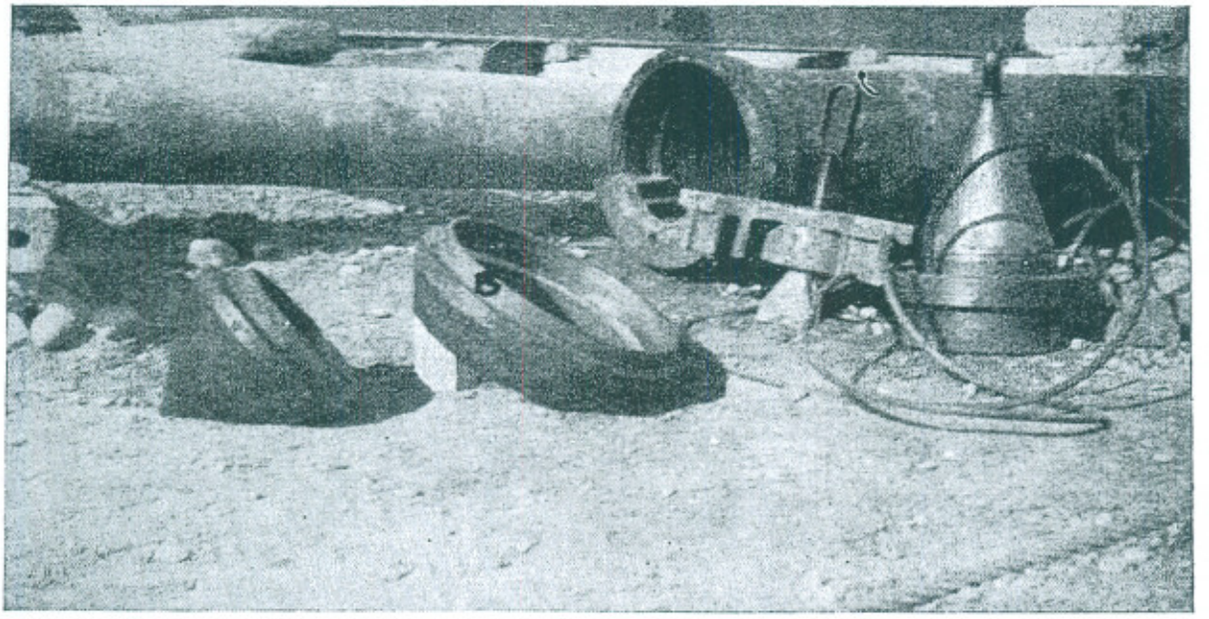
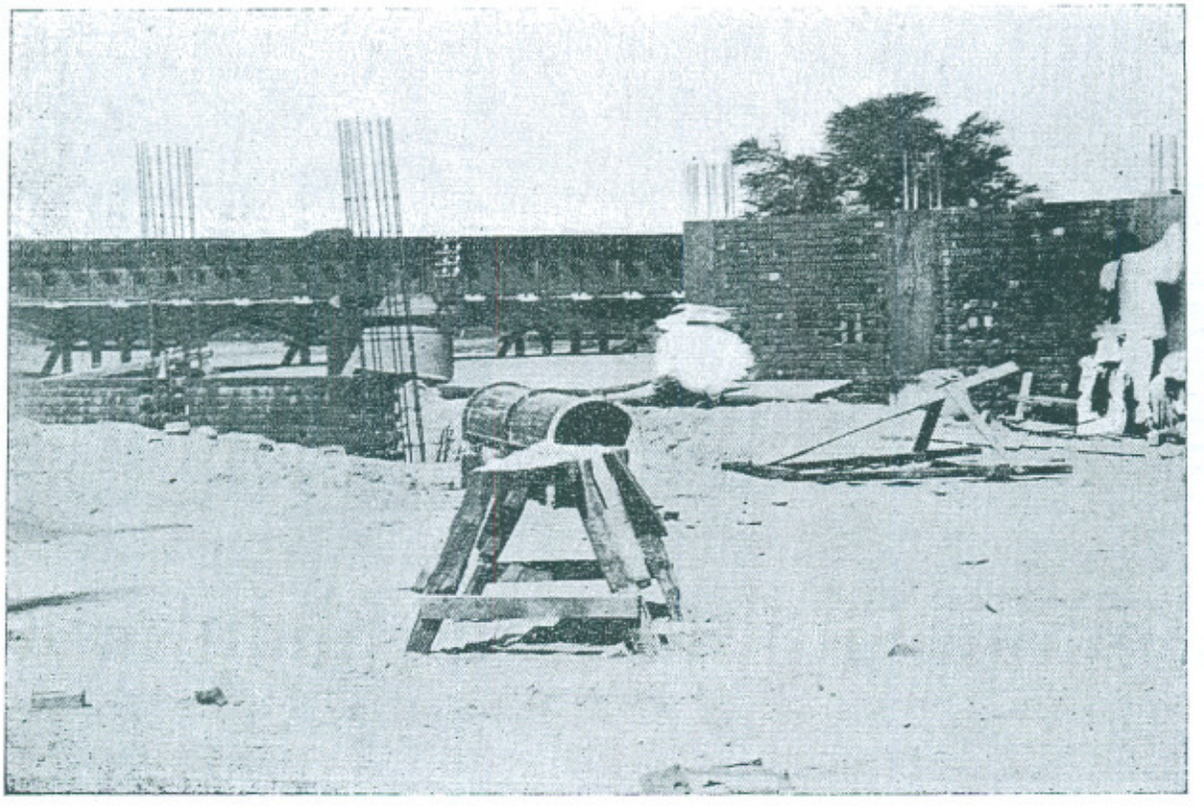


Fig. 8.



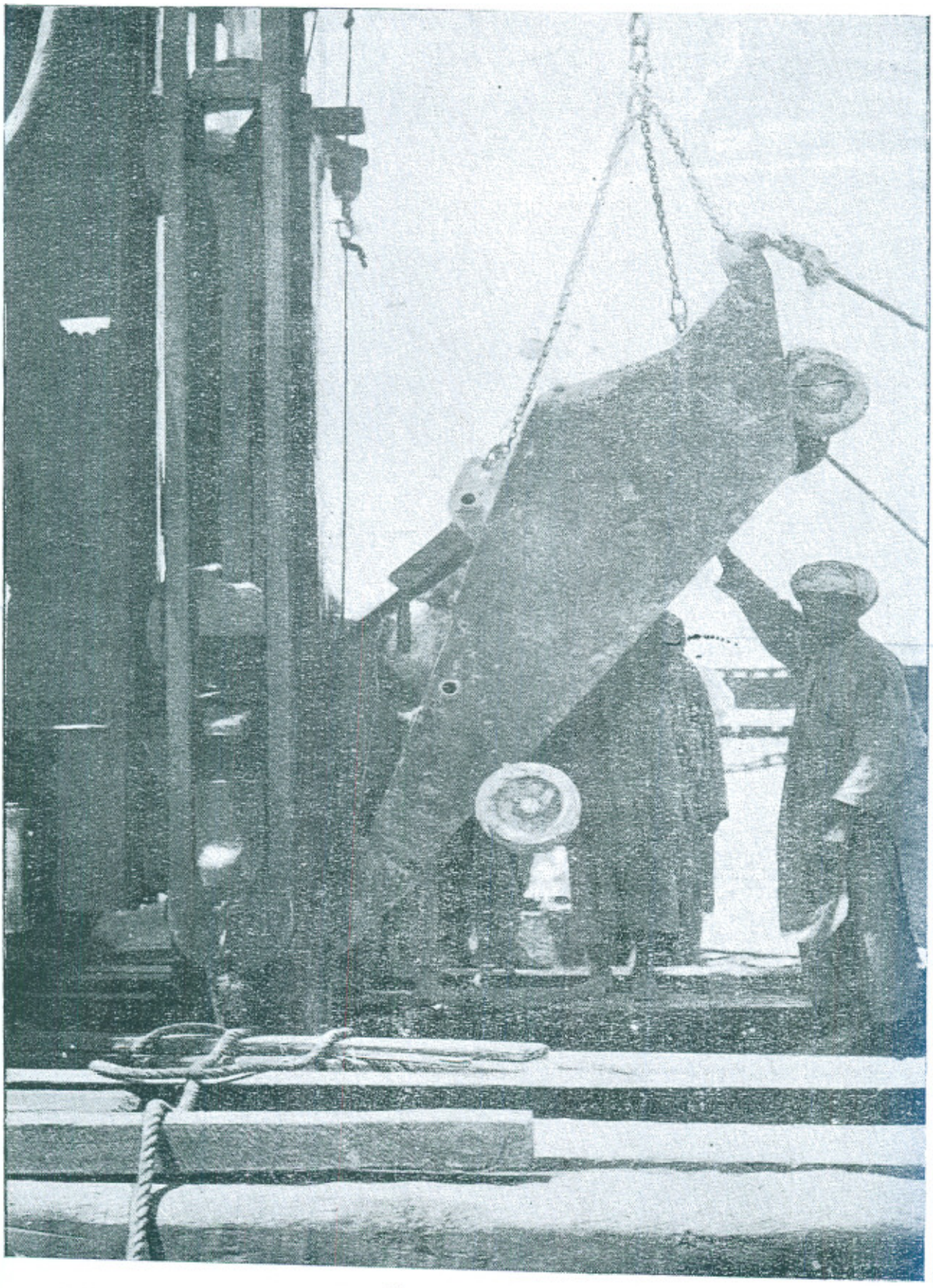
Left. 1. Cast Iron Shoe. *Middle.* 2. Ring Shoe. *Right.* 3. Driving Ring.

Fig. 10.



A thin Pier View showing Column Reinforcement Brick Filling.

Fig. 9.



Concrete Skip.

Fig. 11.

SECTION THROUGH PIER —

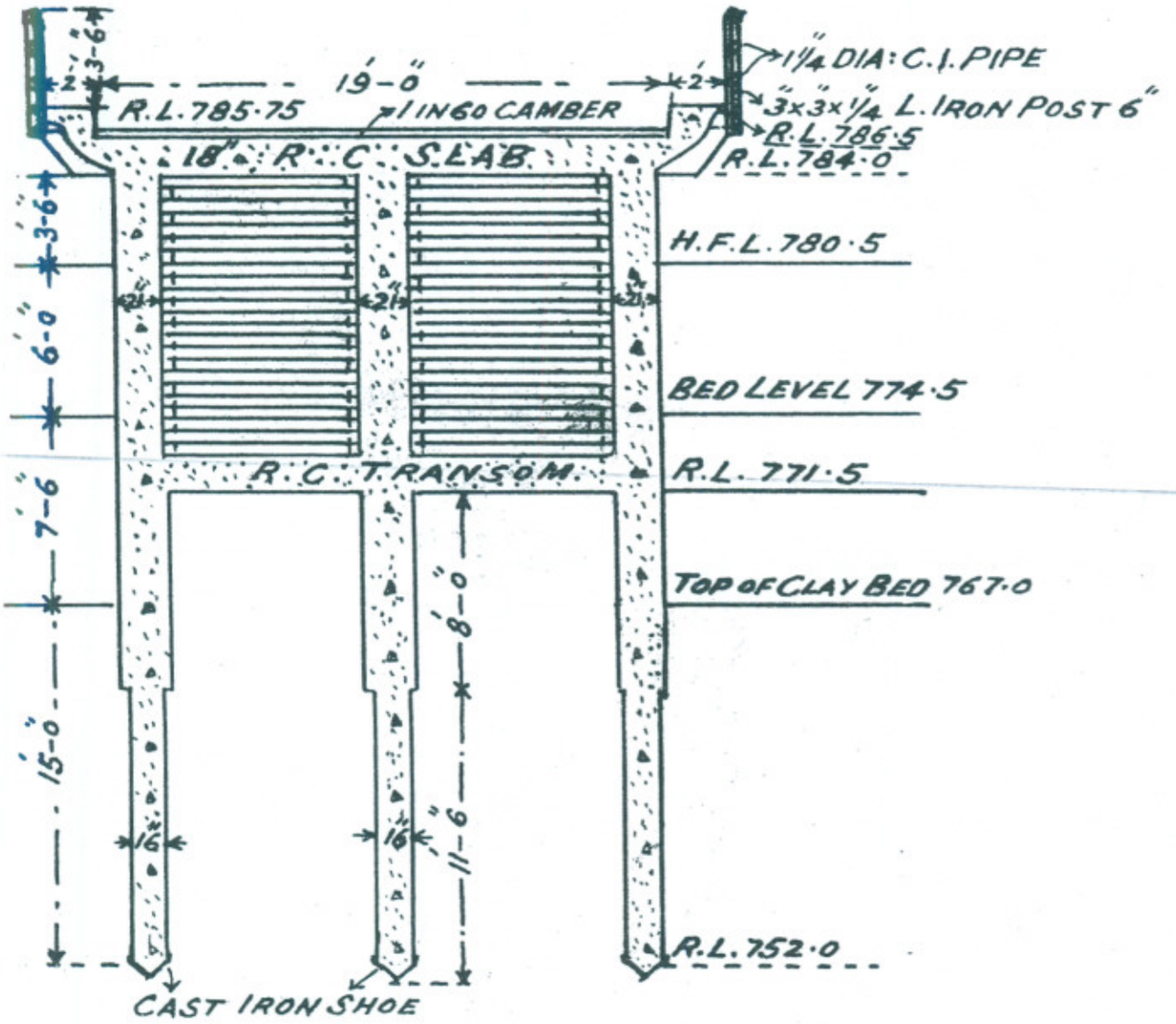


FIG 12.

CROSS SECTION THROUGH THIN PIER

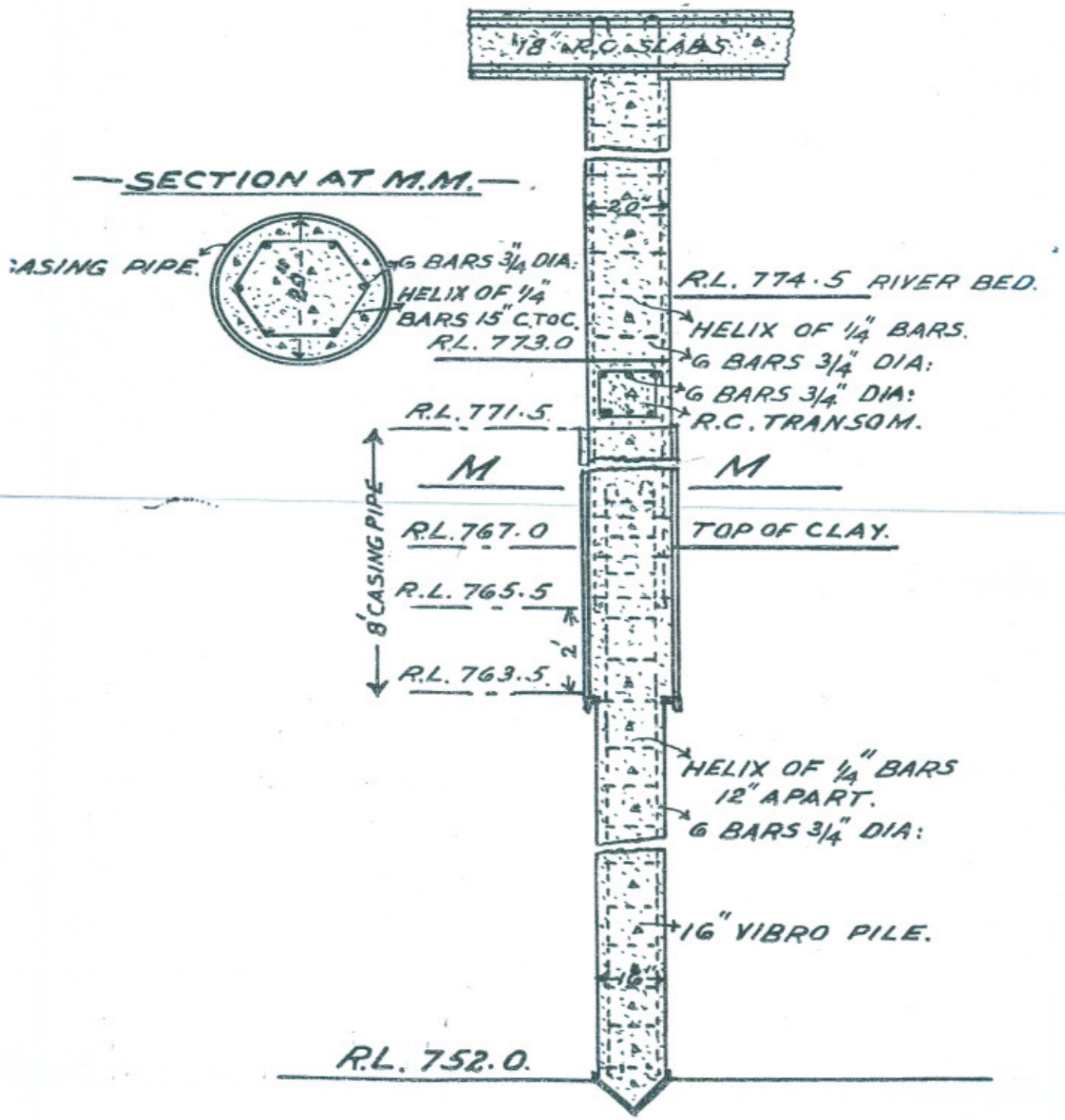


FIG 13.

SS SECTION THROUGH THICK PIER—

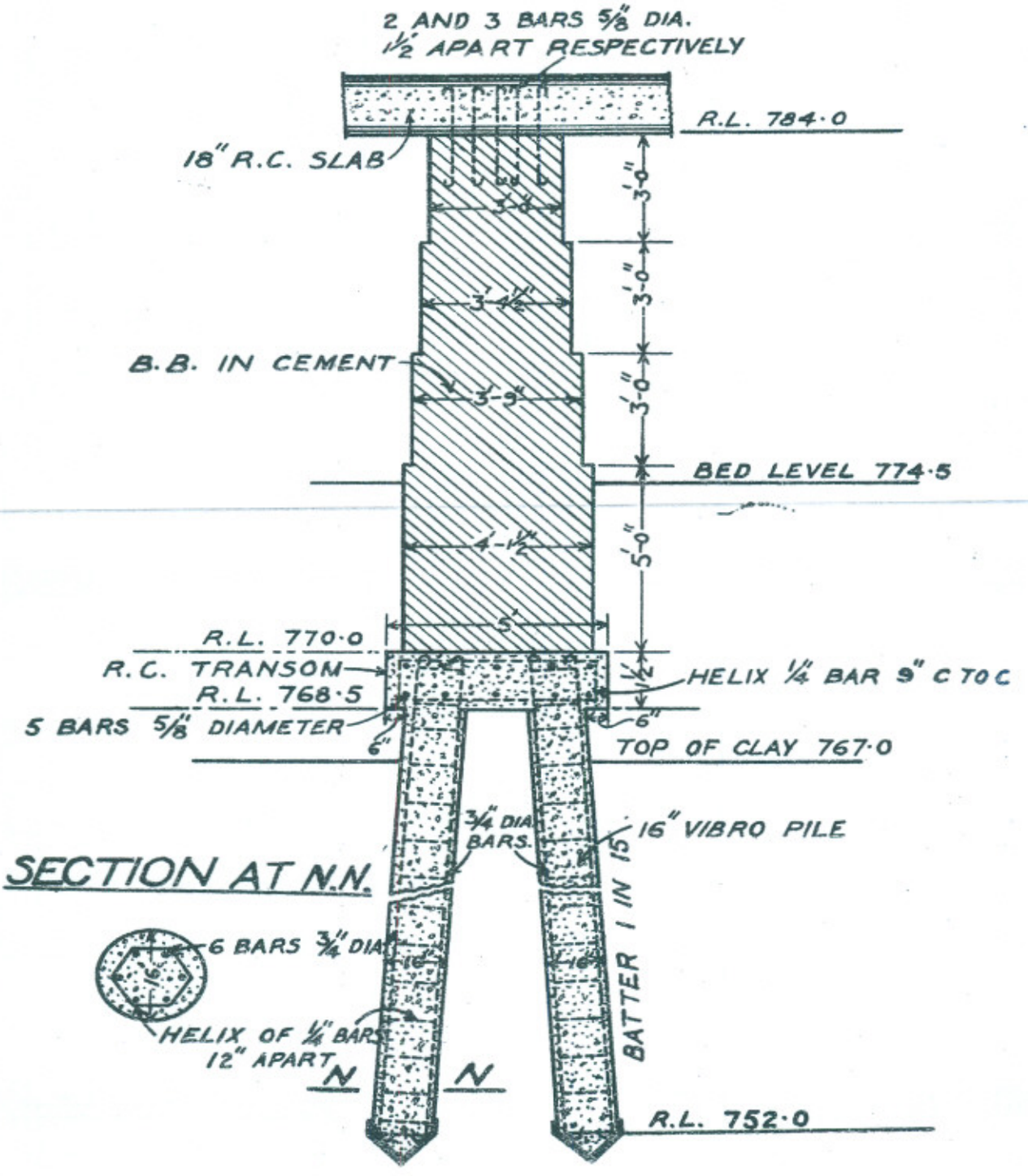
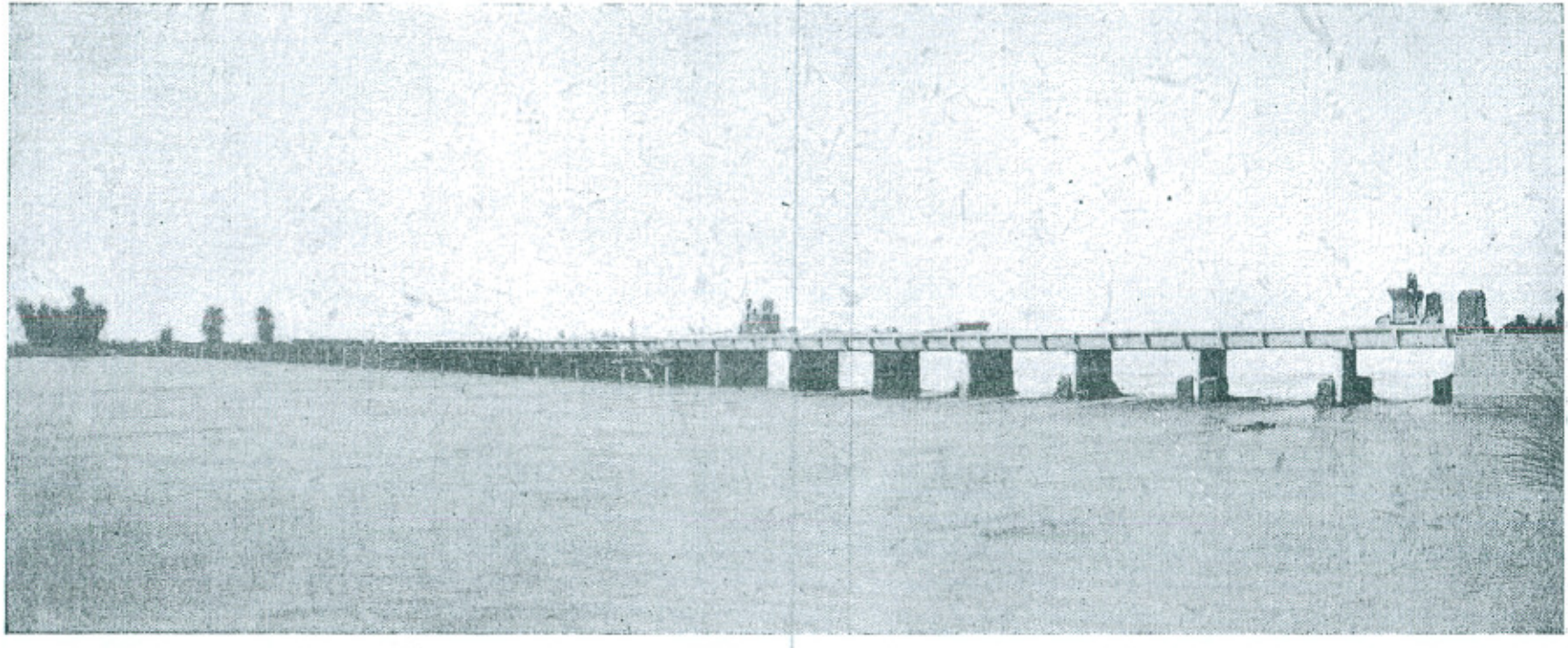


Fig. 14.



View of the Bridge in Floods. Concrete being Poured in II Set of 8 Spans. The First is Complete.

DISCUSSION.

Before introducing his paper Mr. P. L. Varma pointed out that—

(i) On page 137, P. R. was to be omitted.

(ii) On page 141, line 1, instead of figure 10, figure 12 should be written.

(iii) On page 145, line 5, instead of big U, small v should be substituted, and instead of small v, small u should be substituted.

The Author said that in presenting the paper he had made an attempt to bring the unsolved problems of pile foundations in clay, to the notice of the Members of the Congress. In an effort to cut down costs to the minimum, an attempt was usually made to work to the lowest factor of safety, particularly in the design of foundations. A lower factor of safety together with a factor of ignorance of the behaviour of alluvial clays had sometimes landed him in difficulties. The science of soil mechanics—a study of which was as essential as the study of the structural materials—had not been tackled with any amount of success as yet. The subject became of great importance when laying foundations of highways on alluvial clays. A number of experiments on—

1. Mechanical composition of soil,
2. Capillarity,
3. Moisture content, and
4. Colloidal content

of soils had been made in America but the codification of the results had not been possible as so much depended on the nature of the clay. Experiments would have to be made on a large scale in this country to guide the engineer and make his knowledge of soil mechanics definite so that when putting his foundations in alluvial soils in India, he laid his first batch of concrete without any fear as to the result. Any experience on this subject gained by the Members would be of value.

An attempt had been made to show that pile driving formulæ had their limitations when piles were driven in clay. The set of the pile at the time of driving the test pile was no indication of the load carrying capacity of a pile. The tests should be verified by actual loading or re-driving and, if the moisture content of the soil was likely to change, the test should be based on the observations taken under both conditions. Usually the factor of safety under known and definite conditions, based on a precise knowledge of the geology of the ground, might be two, but if the ground was such that the observations were very variable from pile to pile, the factor of safety should be increased to three. In shingle bedded rivers the factor of safety of two never gave unreliable results. Also while making a load test, the fact that a steady load over a long period produced results which could not be seen in short time, should not be ignored.

It would be noticed that all abutments were founded on wells. The mathematical treatment of a group of piles under inclined thrusts or eccentric loading, had, therefore, been omitted from the paper. Occasionally under such conditions, piles in the foundation designed for one condition, showed an upward thrust under another condition of loading. The upward movement was checked by the friction brought into play on the sides of the piles and also by the weight of the surrounding clay. The Author said that his friends in the Irrigation Department had always to design structures against an upward pressure. The Author would be grateful if any light could be thrown on the problem and would like to know if piles had been tried by the Irrigation Department anywhere to take an upward thrust.

Another fact that the Author wanted to bring up was that when putting a new structure in plastic clay (not in sand or shingle) near a Railway Bridge the factor of safety to be used should be slightly increased. The Author always wondered if the vibration transferred to the ground by the traffic on the Railway Bridge at the Bhimber had anything to do with the settlement of the piles. It was apparent that when a pile was driven, the arrangement of the soil particles changed and the vibration from any external source tended to re-orient them to their natural positions which would most probably reduce the value of the bearing capacity of a pile.

Since submitting the paper, the Author said that remarks had been made on the thinness of the piers. A word about the piers would not be out of place. There was no doubt that thin piers possessed no lateral stability but they were so designed that the rigidity of the structure was obtained by making the decking rigid.

Mr. KANWAR SAIN said that Mr. Varma deserved the gratitude of the Congress for bringing forward a paper on a subject about which very little was known and he was sure that this paper would be an incentive to the further investigation of the question of soil mechanics. It was unfortunate that Mr. Varma had given no details of costs in the paper.

Mr. Varma had not indicated in the paper how reinforced concrete caps were designed. Any settlement of the sand underneath the piles would put the entire load on the reinforced concrete caps. It would have been valuable if the Author had given details of the cost of this type of piling and compared it with the cost of steel piles.

R. B. Lala Sant Ram said that the Bhimber Bridge was originally designed by him to consist of large spans of about 45 feet length from centre to centre of the piers, so as to be equal to the Railway Bridge spans on the downstream side. The decking was to be an R. C. cement concrete slab, 6 inches or so thick, supported on R.S. Beams of large size, carried on brickwork piers, 3 feet thick, founded on 6 vibro cement piles driven in two rows of three each. This design was adopted in order to have the spans of the new bridge exactly opposite those of the Railway

Bridge so as to avoid any undesirable obstruction, eddies, etc., which would have been detrimental to the latter. The vibro-pile foundations were adopted with a view to economy in cost, and speed in construction, based on the experience of their successful use on the Wah Bridge near Hassan Abdal. The cost of well foundations was worked out and compared with that of pile foundations and found to be slightly more than the latter even after paying for the loan of the vibro piling plant from the M. E. S. North West Frontier Province, and its transport both ways. After the work was sanctioned on the said design, Captain Jeffries, R. E. of the M. E. S., under whose advice and guidance the use of vibro-pile foundations was adopted in the Punjab, came forward with a suggestion for modifying the design of the bridge with a view to effecting a further saving in its cost. The suggestion was that smaller spans, about half the size of those originally approved, and very thin piers of R. C. concrete supported on two piles thickened at the top, with R. C. concrete construction entirely for the superstructure should be adopted. This would do away with R. S. Beams of large size which would take a long time to obtain. The new design would be about 20 per cent. cheaper in cost, stronger, more rigid, monolithic, and could be completed more rapidly than the sanctioned design. The proposal was adopted, after the Railway Engineers had agreed to the reduced span. The decking was to be a simple slab of about 18 inch thickness, requiring a very simple centring for its construction. It would be cheaper and much easier to lay than one of any other design. Captain Jeffries suggested only two piles under each pier ; but three were adopted so as to be more on the safe side. The stratum of clay, which was found by five trial borings to be present in the bed of the Bhimber was over 25 feet in thickness, lying under 7 feet of sand, and was considered to be ideal for the foundations. Pile driving in it was expected to be a very simple and easy affair. The first indication of trouble came from the fact that the driving was too easy, and the set required for each pile was not being obtained, even at the maximum depth to which the piles could be driven with the plant available. It was, then, considered necessary to bulge the piles at the bottom to gain additional point resistance. Every pile was bulged as described in the paper.

It was, however, only after laying the decking slab of the first eight spans, and the occurrence of a flood in the nallah, which increased the moisture content of the clay stratum, that it was found that the clay was of a very plastic nature, and behaved in a way entirely different to expectations. It was evidently not impervious to water, and what was worse, retained the water much more than other kinds of soil. This resulted in the skin friction, developed at the time of driving, being considerably reduced.

The soil was neither very permeable like sand or gravel, nor quite impermeable like stiff hard clay. It was midway between the two and, therefore, had the bad points of both. The increase of moisture acted as a lubricant and the lubricating effect increased after movement due

to vibrations. It is quite possible, as the Author had remarked, that the effect of vibrations from trains passing on the Railway Bridge nearby may have had something to do with the settlement.

The steps taken to give additional support to the pile foundations were described by the Author in the paper and need not be discussed further. The object was to provide as much additional bearing area, to take the full load expected, so that the factor of safety would at least be doubled.

The saving in cost expected to be effected by the adoption of the modified design, was lost by the subsequent modifications. However, although the failure of the pile foundations in this case caused the Engineers-in-charge some very anxious moments, he thought the experience gained would be very useful to the Engineering Profession. The main object of the paper is to stress the necessity of basing calculations of the load-bearing capacity of piles on actual observations and tests of the soils to be dealt with under different conditions, rather than on formulæ based on experience gained elsewhere, to stimulate research on the subject, which is entirely novel to the profession in India and, therefore, extremely interesting.

Mr. S. L. KUMAR said that the Author had stated in the paper that the foot of the piles was 22½ feet below the bed level of the Bhimber Nala at the site of the bridge. The piles having failed they had provided R. C. circular footings round the two end piles and a masonry well round the two end piles and had noticed from the train the previous day that a pitching stone protection floor was now being laid round the piers of this bridge. Mr. Kumar asked the Author why this additional protection was found to be necessary and whether on the whole the simple well foundations with masonry piers as used by the railway on their bridge close to this bridge was still not the best and in the long run a less expensive method of constructing a bridge.

In reply to the discussion Mr. Varma said that Lala Kanwar Sain had asked him to explain why expansion joints had been kept 160 feet apart. It had been mentioned in the paper that although the piers used were thin, the central pier was made very thick, and the slab tied to this pier. The provision for expansion was therefore actually wanted for a length of 80 feet only, the slab expanding on either side from the central thick pier.

Mr. Kanwar Sain doubted the suitability of a 18 inch thick slab as against slabs and T-beams. This point has been explained by R. B. Lala Sant Ram. Simplicity in construction and cheapness were the chief aims.

Mr. Kanwar Sain had mentioned the side friction. The whole purport of the paper was that side friction was a variable item in clay soils and the set of the piles as obtained by a hammer was no indication of the side friction under stationary conditions later on.

The reinforced concrete caps were designed against the total reaction from the soil due to the dead load of the superstructure. Lala Kanwar Sain had pointed out one omission which should have been brought out earlier by the Author. Contrary to the suggestion made by him that on slight settlement the entire load would be transferred to the caps, the Author thought that settlement in the foundations would increase the carrying capacity of the piles by compressing the soil around them. The caps would not take any load unless the settlement occurred and, as remarked in the paper, every one of the foundations was tested for a load equal to double the load that was likely to be imposed on it. Slight settlement did occur which was considered necessary in order to make the footings take up their share of the load. Contrary to Lala Kanwar Sain's remarks, the Author anticipated that after settlement the carrying capacity of a pile would be increased and the footings relieved of some of the load on them.

Regarding Mr. Kumar's remarks the Author said that he did not know why the foundations were being strengthened against erosion as the work was not in his charge. He did not consider any such work necessary. Well foundations were not adopted for the reasons explained by Rai Bahadur Lala Sant Ram in his remarks.