Pylon foundation of four cablestay bridges: the Indian experience

Autor(en): **Subba Rao, Tippur Narayanarao**

Objekttyp: **Article**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

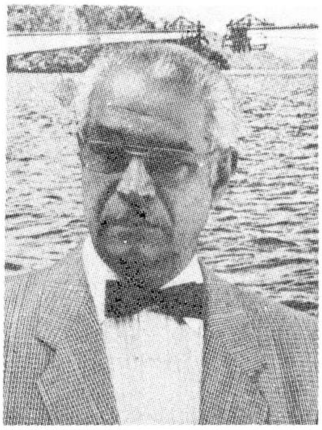
Band (Jahr): **80 (1999)**

PDF erstellt am: **27.06.2024**

Persistenter Link: https://doi.org/10.5169/seals-60754

'PYLON FOUNDATIONS OF FOUR CABLESTAY BRIDGES *THE INDIAN EXPERIENCE*'

TIPPUR NARAYANARAO SUBBA RAO
Chairman
Construma Consultancy Pvt. Ltd.
, Bombay
India



T. N. SUBBA RAO born 1928, Graduate : Univ. of Mysore, Hon. Doctorate : Univ. of Stuttgart, Fellow : Indian National Academy of Engineering; Past Vice-President : FIP & IABSE; Past President : Federation Asian & Western Pacific Contractors Assn; Managing Director (Retd.): Gammon India Ltd., Premier Design & Construction firm; Presently Chairman : Construma Group, Consultancy firm for Civil Engineering Works.

Summary :

The Paper deals with the piers and foundations of four cable stay bridges, each of which are significantly different in concept and execution. Two of the bridges have been executed. One has been executed with the described concept but for a different structural layout and the fourth one is about to be taken up. The kind of problems met with or expected to be met with, are dealt with in some detail. The major project executed, namely the Hooghly River Bridge in Calcutta has been given more attention because of the large span and complexity of the foundations in alluvial soil. The subjects chosen bring out the fact that the foundations for cable stay bridges require indepth attention because of the critical interacting behaviour between the foundation and superstructure system.

Introduction :

The adoption of cable stayed bridges for long spans, generally exceeding 200 m, demands a careful assessment of the characteristics of the soil under the foundations, more rigorously than is normally practised for other types of structures. The reason for this detailed investigation is essentially to ensure that the soil through which the foundations are taken down, as well as characteristics of the soil at founding level, are such that under load, settlement, tilts and shifts of the foundation, the stresses are within acceptable limits and sufficient lateral stability can be realised. It is evident that occuring of a slight tilt or movement of the foundation under a pylon pier could lead to a substantial and undesirable redistribution of forces in the superstructure. The sensitivity of the structure to the foundation behaviour, is therefore an aspect which cannot be overlooked from the stand point of choosing the parameters for foundation design, and the type of foundation most suited for the location. This observation also applies, though not as intensely, but nevertheless without loss of importance, to the design of the piers supporting the superstructure at either end of a cable stay unit.
The several types of solutions adopted for structuring a cable stayed decking, naturally results in the forces on the end piers and the intermediate pylons being somewhat different in terms of the quantum of the force distribution for the same deck module. Depending on the ground configuration, the transverse and longitudinal forces could be transferred by option, and to the

extent desired, either through the pylons or through the end piers, as convenient. The gravitational loads of the structure are by and large transferred to the foundations through the pylons and this constitutes a large percentage of the total force on the pylon foundations.

1.0 SECOND HOOGLY BRIDGE :

1.1 General Features :

The foundations and the piers of the Second Hooghly Bridge in India with a main span of 457 M and side spans of 182 M are most unusual and envelope conceptualisation of several types of complex foundations met with in practice. The (Figs.1&2) illustrate the general layout of the bridge and of the piers and foundations in some detail.

The structural system of the deck consists of portal type pylons with a through deck, with provision of restraint bearings in the longitudinal direction over pier 1 and free movement bearings at pier 4. The composite deck is transversely supported over these piers through the provision of lateral bearings.

The bridge is close to the sea and apart from the substantial range between low and high water levels, the standing wave from the tidal bores sweeping up the Hooghly quite often measure upto 2.5 M in height and need careful reckoning.

The bridge is located in a seismic area and is designed for a seismic intensity of G/15

The area being prone to cyclonic storms, wind force corresponding to wind speeds of about 200 Km per hour are to be expected.
The main pylon foundations 2 & 3 are also designed for the impact of floating vessels of 10,000 tons displacement with an approach velocity of 1.5 Knots per hour.

1.2 Choice of Caisson Geometry and Sinking :

Considering these factors and the Gangetic terrain conditions of the soil, the type of foundations chosen under the pylons consisted of two circular caissons each having 9 compartments, interconnected by a very rigid pier, to provide an effective transverse portal system. The top of the caisson is kept just above the lowest water level but piers extend nearly 12.4 M above this level.

The cellular caisson layout is dictated by that of the pier, which is also cellular. The forces from the pier are transferred directly into the inner walls of the caisson parallel to the pier, and the forces thus transmitted, are carried through these walls over the entire plan area of the caisson, almost immediately below the base of the pier. The internal layout of the caisson walls give the caisson a very rigid structural system; the force transfer path to the soil is shortest as also concentric to the caisson. The transverse portal action is complete and effective and provides in that direction a high level of security against the action of transverse forces, impact from floating vessels, forces generated due to presumed differential settlement of the twin caissons below each pier and the like.

The caissons have been designed to be empty throughout their working life in order to reduce pressures on the soil. The Gangetic soil has clay bands interspersed with coarse and fine sand layers and thus invites settlement threat over the years. Since the strata is sedimented uniformly and characteristic over a wide flood plain, the likelihood of differential settlement between the caissons is remote and should any such phenomenon manifest itself, the transverse monolithic behaviour of the caisson plus pier system would counter it by seeking a new equilibrium status.

In the longitudinal direction, the forces transferred from the pylon act on the total pier caisson system as a free cantilever, with the force on each base under the twin legs of the pylon being somewhat different, especially under seismic and wind load conditions. The stability of the system is checked as a rigid mass founded on an elastic soil.

The settlement of the caisson pair is evaluated taking the specific alternative bands of sand and clay layers into account. The settlement of both the pylon piers 2 and 3 are not expected to be substantially different and therefore the structure is unlikely to be geometrically disoriented. The section of the caisson in plan, is checked for the non-uniform soil pressure distribution around it under various resultant loading conditions, and has been accordingly reinforced.

The twin caissons 2 being located alongside the bank, are sunk from a dry platform formed with the help of steel sheet piles, whereas the pair of caissons 3 has a double walled steel shell prefabricated in a dry dock. The shell 3 was slipped at high tide into the river, brought to location, filled with tremmie concrete to settle on a pre-formed flat river bed, and progressively sunk through the soil by open grabbing inside the cellular spaces in a systematic pre-ordained manner. A very close watch was maintained to account for the possibility of sudden scour below the shell occuring as a result of tidal and river current forces. The caissons were floated one after the other and the downstream one was placed in position after the upstream one had gone sufficiently deep, so that the possibility of suction of the soil from one caisson to the other was minimised. Once both the caissons were placed in position, they were taken down systematically with a level difference not exceeding 4 to 5 M, until they reached their final depth,

1.3 Caisson Plug and Cover Slab :

After reaching the final level, the twin caissons 2 were plugged with prepacked concrete and caissons 3 were plugged with tremmie concrete, as a first stage operation. The difference in the plugging method adopted was essentially to check the performance of the two methods. Both proved effective.

To avoid the possibility of seepage of water from the conical interface of the plug and the caisson shell, the caissons were dewatered after plugging and pier construction, and a reinforced concrete slab notched into the caisson walls, scaled each cell of the caissons. At the same time, the first stage plug was checked, qualitywise and water tightnesswise. The safety against buoyancy under this equilibrium state was 1.25. The plug in each caisson is checked for the reactive forces coming from the soil for the buoyant weight of the caisson including partial pier weight, before the reinforced concrete slab is concreted over it to form an integral part of this plug. Under service load conditions, on completion of the bridge, the integrated plug is checked for the highest reactive forces from the soil caused by the most severe loading combinations. These forces are transmitted by the plug to the inner and outer walls of the caisson and the bottom-most section of the peripheral wall has been reinforced for the bursting and bending forces coming on it. A finite element analysis for both the first stage plug and the integrated plug generally indicated good dome action and effective transfer of forces from the caisson to the soil. The plugging being a very critical activity, had to be performed with much care and pre-planning, so that the plug and the caisson behaved as if they were an integrated structural unit, capable of withstanding and transmitting to the soil in the most appropriate manner, the very large forces imposed by the structural system. The cells in the pier are interconnected by an opening above the plug level, to facilitate access and inspection.

The slab capping the caissons at near lowest water level is rigid enough to energise the entire section of the caisson almost immediately below the soffit level. For concreting the cap and pier, caissons 2 were garlanded by a sheet pile cofferdam on the river side and posed no problem. However, for concreting the cap, the caissons 3 located midstream were provided with a circular steel cofferdam reaching above high water level and 7.5 M high. It incorporates a gate mechanism along the transverse axis, (Fig, 2) as an expedient for pier construction..

1.4 Some Factors Affecting Caisson Sinking :

Great care had to be taken during the sinking of caissons to see that they are sunk almost vertically in their true position. This is to avoid eccentricity of the pier over the cellular walls of the caisson beyond accepted limits, and also to ensure that the pylon is located directly over the central pocket of the caissons.

A shift of ½% (2.25m) of the central span inbetween the caisson pair 2 and 3, with corresponding span variation was acceptable and designed for. However, careful sequential sinking and precautionary measures like maintaining the water level inside the caissons higher than the river level with a view to prevent sand blows during sinking and others, helped in reaching a main span variation of less than 1 M and transverse axis variation of less than 0.7% of the caisson diameter. The latter control ensured that the pier walls rested directly on the corresponding cellular walls of the caissons to enable the flow of forces effectively from the pier to the caissons.

To meet any accidental sand blow conditions during sinking, the caisson was designed for external earth pressure acting on half the diameter on the caisson, with consequent bending effect on the caisson in plan. The vertical steel was also checked for a sudden de-pressurised condition that may develop upto half diameter above the cutting edge of the caisson during the final sinking process. (fig. 1)

The caisson being massive, exhibited least sensitivity to movement and so long as the grabbing inside was systematically executed, sunk slowly without causing much anxiety.

In view of the depth at which the caissons are founded below high water level, the need for pneumatic sinking, should the sinking be obstructed by sunken boats or logs or other material in its path, was anticipated. The steel in the caisson is catered for this emergency so long as it occurred within a depth of 35 M. However, if such a case arose beyond this limit, the solution lay only by sending divers to cut the material underwater and remove it, which would have been a slow process but nevertheless imperative. Luckily, except for a 5 ton anchor left behind by ships, which came up during the grabbing operations, no other problem was faced. This contingency should nevertheless be anticipated and provided for in the design.

A garland of fenders for absorbing the impact of floating vessels is fixed on pier 3 only since the draft at pier 2 will not normally allow vessels to come close to it. Nevertheless, provision is made to fix fenders at this pier as well should a change in river behaviour necessitate this. This has enabled the real force on the pier itself to be brought to a manageable level.

1.5 Piers 2 and 3:

The execution of the piers, starting below the water level and above the concreted cap required as a pre-requisite, definition of the exact location of the caissons in relation to their designed position and thereafter adjustment of the transverse axis of the pier in the direction of the span, so that the continuity between the transverse walls of the caisson pair and of the piers was realised within prescribed tolerance limits. Because of the presence of the aforestated sheet pile cofferdam, executing pier 2 posed no problem.

However, pier 3 required a very different treatment as no such cofferdam was feasible. The work on this pier starting below the water level, required it to be split into two parts. As first part, the pier and caisson cap portion within the cofferdam was constructed upto 9.0 M height (+ 7.0 M) of a total 14.0 M including the cap depth; This level was above high tide level experienced during the season. Provisions was made in the cap and pier to integrate balance portion of the pier between the caisson pair at a later date. The transverse reinforcement in this section is necessarily very heavy, due to portal action of the caissons plus pier, and demanded very meticulous layout and positioning.

The intermediate section of the pier was cast at 7.2 m level over a supporting platform suspended from a steel truss spanning between the piers already cast within the caissons. (Fig. 2) The lowering system was very carefully detailed so that control of any one of the 4 points of suspension was possible independently of each other. At either end of this intermediate section of the pier, a sealing arrangement was incorporated to prevent ingress of water from the soffit, when the section was lowered to its final position below water level. The lowering activity presented no problems since all contingencies were anticipated and provided for. Especially important was the exact positioning of the reinforcement along the pier axis, so that when lowered in position, it matched perfectly with the steel provided in the portion of the pier partly concreted over the caissons. This entire performance required design, detailing and construction management skills of the highest order.

Before lowering this precast pier section, the afore mentioned gates in the cofferdam were removed and this allowed water to food inside the cofferdam over the well cap. After the section was lowered, gates were inserted spanning the gap between the two halves of the piers on either side and this enabled dewatering the pier section between the gates. Much care was taken in detailing the junction between the gates and the soffit seal; It was a very vulnerable joint. The small leakage witnessed was sealed by divers with quick setting micro concrete during low tide. Following cleaning of the projecting reinforcement, the balance portion of the caisson cap and the piers upto + 7.0 m were concreted in the dry and thus the entire pier became integrated as one unit. The remaining portion of pier above

7.0 m was gradually raised upto its full height thereafter, in two meter lifts covering the entire plan area of the pier.

Of particular relevance is the concreting of the pier portion where the anchors for the base plate of the pylons are located. This required accurate positioning of the anchors with the help of a template. The reinforcement in the anchor block was so detailed that it distributed the forces to the walls of the pier effectively through shear. The tension forces caused on the pierhead by the 3.5 degree transverse inclination of the pylon is countered by prestressing.

The meticulous planning and execution of the pier in two parts and integrating the central unit below water level, the positioning of the anchor plates and several other activities, required most careful attention to detailing to facilitate reinforcement placement, avoid unacceptable crack widths, ensure sufficient cover and concrete compaction for durability and other factors consistent to obtain a totally integrated pier and caisson system.

1.6 General Issues Concerning the Caissons :

The caissons were filled with water soon after the plug cap was laid and prior to the integration of the pier section, to enable major settlement of the foundation to take place. Following pier integration, the caissons were dewatered completely, the silt which had collected over the concrete plug was removed, the laitence though hard chipped out, the surface cleaned efficiently and the concrete slab referred to earlier laid over it. Thereafter, they were checked for any leakage of water and where it occurred though to a very small extent, was plugged by injecting cement or epoxy grout.

The caissons 2 and 3 were again filled with water, and progressively dewatered with superstructure load buildup and are designed to be **kept empty** throughout life. Under this condition as the caisson walls have to withstand large horizontal forces form the soil, a plane frame analysis of the caisson geometry in plan was carried out and the stress levels checked.

A periodic check is maintained as regards progressive settlement of the caissons and the scour around them. A SAP IV programme carried out for the main pier cum foundation system 2 and 3 for the severest lateral and vertical forces, indicated that the system behaved as a single bay portal frame with a hinged base and the effect of differential settlement of the order expected between the pair of caissons supporting each pier would be small. A gross settlement of 200 mm and relative settlement of 25 mm between the pair of caissons was part of this investigation.

1.7 Special Design and Conceptual Issues

Briefly, they are :

- The assessment of risk factors arising out of the choice of soil parameters and the force levels due to seismic, wind and river current.

- The structural system best suited for absorbing both the very large longitudinal and transverse forces, inherent with large spans.

- The type of foundations to be adopted for the pylons, so that they suffered to a minimum extent due to settlement, without sacrificing integrity of their behaviour and performance.

- The need to keep the caissons dry to reduce foundation pressure and consequently stipulation of a crack free design for the outer walls of the caisson.

- The methodology for integraing bottom section of pier 3 below water level keeping risks and durability factors in mind,

- The possibility of scour occurring during construction and causing the caissons to be shifted from their true positions.

These issues highlight the innovative approach and the close interaction required between the design and construction teams responsible for the execution of the foundations.

2. AKKAR BRIDGE :

The bridge over River Rangeet at Akkar in India has a central pylon with a span of 79 M on either side. The pylon and the deck are all constructed in concrete. (Fig. 3).

The pylon is located on a single circular caisson sunk through rock by using controlled blasting techniques. The caisson is shielded against blasting shocks with steel upto a height of 4.15 M on the outside and 7.70 M inside the dredge hole. The concrete pylon rests on wedge shaped hollow pier just above the high flood level. The nosing of the pier on the upstream side is shielded by armour-plates to protect it from the impact of huge boulders rolling down the fast flowing river during high floods. The pier is founded on a thin slab capping the caisson. (Fig. 3)

The wedge shape of the pier causes the load from the pylon legs to cause a splitting action at cap level. This is accounted for by closely spaced small dia. reinforcement. No prestressing is applied. As the well cap is too thin to help even redistribution of load from the pier over the caisson ring, heavy hoop reinforcement is provided in the caisson just below the cap, to counter the splitting action caused by the pier, as also to help gradual dispersion of the load into the steining; It is as if the pier sprouts from the steining as an integral part of the pier caisson system.

The deck is freely suspended from the pylon. Unlike the Hoogly Bridge both the transverse and longitudinal forces caused by wind and seismic action (G/10), temperature and braking effects are taken equally at both ends of the bridge, by an abutment block, interfaced by a multiple neoprene bearing arrangement. The vertical forces are carried by horizontally placed neoprene bearings and the abutment block itself provides counterweight in an emergency. The abutment is keyed into the foundation rock through shear keys.

The bridge site with steep banks and good foundation rock on either side is excellently suited for absorbing the deck forces in-plane and the abutments were designed to suit. The positioning of the neoprene bearings to absorb longitudinal forces, transverse forces, and downward loads together with provision for their inspection and replacement, constituted an important criteria while defining the layout and geometry of the abutment. Again, the geometry of the deck penetrating into the abutment was such as to provide access to the back-stay cables, which may require restressing in future.

3. JOGIGHOPA BRIDGE :

This proposed (but not implemented) double deck rail-cum-road bridge for dual line broad gauge track across the Brahmaputra river in India has a cable stay span for a part of its length, with spans of 138-348-138 M. The superstructure is designed to be of steel construction. (Fig. 4)

The solution for the caisson foundations of this cable stay module presented different problems because of the presence of slopping rock strata. As a result, one pylon and one anchor pier caisson foundation are founded in sand, approx. 65 M to 70 M below the bed, and do not pose difficult problems for sinking and plugging. The other pylon and anchor pier caisson foundation close to the bank have problems in founding, although not with regard to their initial sinking in the sandy river bed. The anchor pier also absorbs the longitudinal forces and it is located and anchored into rock 35 M below the lowest water level. The anchoring of this foundation into rock under pneumatic conditions, although hazardous at that level, can nevertheless be accomplished by a planned and systematic excavation of the rock. However, the same solution cannot be applied to the pylon foundation, which encounters the slopping rock approx. 45 M below the low water level and is thus beyond safe pneumatic sinking limits. Since the scour in the river extends upto this level, the need for effectively anchoring this foundation in rock is a vital necessity.

The layout for all the foundations consists of a Double D caisson. The piers are founded at water level on a stiff cap, which redistributes the forces to the caissons most effectively.

The caisson of 28 M dia. and with a twin dredge hole has pre-formed circular openings of 1.5 M diameter at 5 M centres within the steining of the caisson. These openings are filled with sand during the sinking process and once the caisson touches the rock level, the sand is washed out. To avoid tilt of the caisson during its final sinking process close to the rock strata, as also to prevent this occurring by the cutting edge touching the rock accidentally by sudden sinking in the last few meters of sinking left, the caisson is stopped short of the rock level. Divers are then sent to stabilise the caisson by providing chairs from below to support it. The soil at the founding level is thereafter cemented by injecting cement grout both inside the dredge hole and outside. Holes of 3" dia kept at 2m intervals in the steining close to the outerperiferi, with exit holes in the curb close to the cutting edge, effectively carryout this grouting operation. The compacted soil at the base further stabilises the caisson. This soil and the rock strata is then bored upto 3 m. through the 1.5 m dia openings and concreted upto the top to establish a good anchorage for the caisson. The stub piles are stressed vertically into the rock mass to establish a positive anchorage and help accept both over turning and shear at the rock level.

The next stage activity covers the cleaning of the dredge hole in the caisson, assisted by divers, followed by plugging with concrete under water upto three quarter the height of the final plug thickness. A central circular opening in the plug is maintained upto rock level during this first stage plugging operation. The caisson is dewatered after plugging, the referred opening is cleared of all silt, and the portion excavated and concreted upto a minimum depth of 1m in rock. This provides a good shear key to the system against sliding on the slopping rock and relieves the shear being felt by the piles. This central

concrete fill is carried above the plug to the full designed height of the plug and integrated with it with dowel bars.

The final outcome of this construction system is that the piles take vertical forces, the central key takes the shear and the vertical prestress aids the anchoring system. Some variations to this system by way of providing a pier cap over the plug, or adopting other means to grout the soil at the founding level, etc. are indicated.

This concept has been successfully tried out on the circular caissons, supporting 120 m structural steel spans, finally adopted for this bridge.

4. BAGCHHAL BRIDGE:

This bridge designed to span across the river Sutlej in the back waters of Govindsagar Lake in Himachal Pradesh, has perforce to be designed as a cable stay bridge with an imperative main span of 184 m and side spans of 75.45 m, thus providing a total length of 234.9 m between abutments (Fig. 5).

The pylon is in concrete and is founded on open foundations in rock strata. The shape of the pylon appears elegant and is designed to reflect the hand clasped *'Namaskar'* concept cradling the concrete deck in between. The deck is 12m wide, carries a 7.5m dual lane carriageway for Class AA & 70R loading, flanked by 1.5m wide footpaths, with railing and crash barrier protection.

The cablestays supporting the deck are anchored in a pylon head above 32 m ht. along the central axis of the bridge. The A frame and the lower V frame in reverse are anchored to a cellular pier, having openings to allow ingress of water and prevent one sided water pressure. The reason for choosing the cellular pier lies in the fact that the transverse contour of the hill on either approach, has a steep fall and a A frame solution, though simplier, would have necessitated unequal legs of substantial height. The pier is founded on a footing which in turn rests on a 1:3:6 concrete foundation. (Fig. 6)

The bridge has a rise of 1.80 m across the main span and the curve is tangential at the pylon point with a straight approach to the abutments. The deck is integral with the pylon and the temperature effects are substantially absorbed by the arch effect and low stiffness of the deck in the main span. The expansion of the 75.45m length is accommodated at the abutment end. Since the pier and deck are integral, the design is complex but greatly improves the global behaviour and stability of the pylon. The pylon and deck are checked for accidental failure of one cable with designed load traffic and also for controlled one lane traffic during replacement of one cable. The pylons are designed to cater for the entire wind/seismic effects on the deck plus its own system behaviour. However, as a measure of caution, transverse and longitudinal forces are also designed to be absorbed to the extent of 35% at the abutments. The deflection of the pylon along the Longitudinal axis is also accounted for in the design of the expansion joints at the abutments.

Work on this project is slated to commence this year.

bridges causes the decision much more risk oriented. The very size of the foundations to carry all the loads of such large spans, again demands a much more detailed assessment of the forces coming on them and calls for computer aided finite element or simialr techniques, to assess the stresses coming on the critical components of the foundations. Careful detailing without sacrificing integration of the different components, consistent with the construction sequence adopted, is a cardinal requisite. This seeks pre-determination of the exact construction methods to be programmed, as well as a very close interaction for realising the scheme, right from concept to practical reality, among all concerned.

Credits :

Credits are due to Hooghly River Bridge Commissioners; M/s. Schlaich Bergermann und Partner, Germany; M/s. Freeman Fox Limited, U.K.; erstwhile colleagues at Gammon India Ltd., and Associates at Construma Consultancy Pvt. Ltd.

5.0 CONCLUSION :

The problems of foundaitons of cable stayed bridges are not unlike those met with in the design of other types of bridges. The cited foundation for Jogighopa bridge resting on sloping rock and beyond acceptable pneumatic sinking limits, would be the same were the bridge cable stayed or otherwise. However, the very large spans adopted for cable stayed

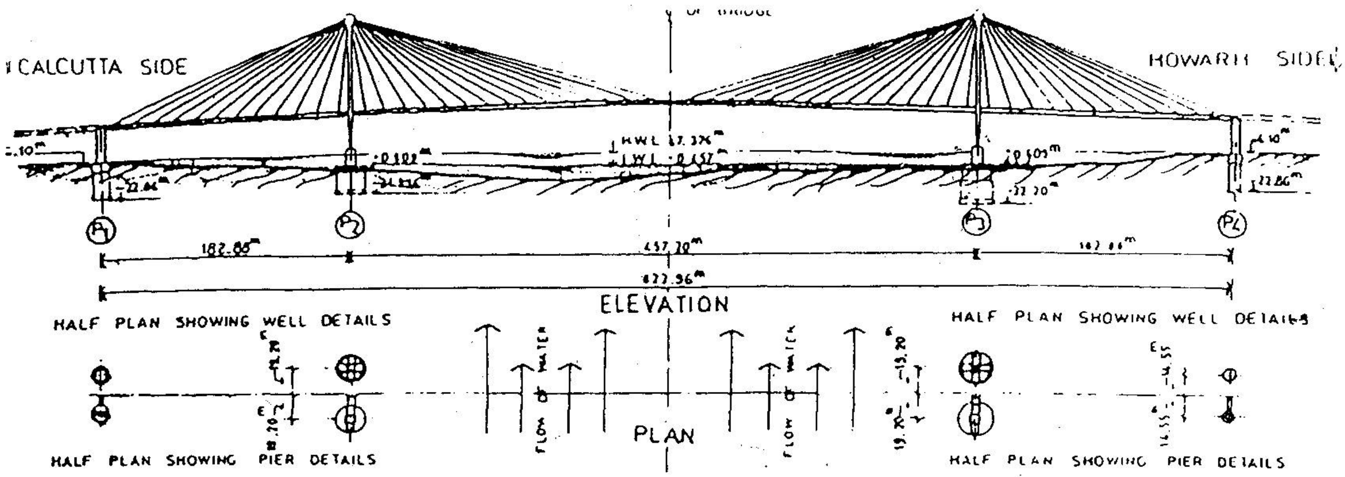


FIG. 1 GENERAL ARRANGEMENT

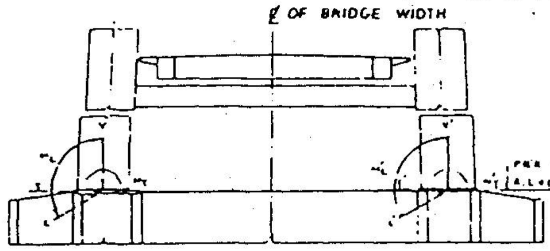


FIG. 2 DIRECTION OF LOADS FORCES AND MOMENTS AT THE PYLON BASE

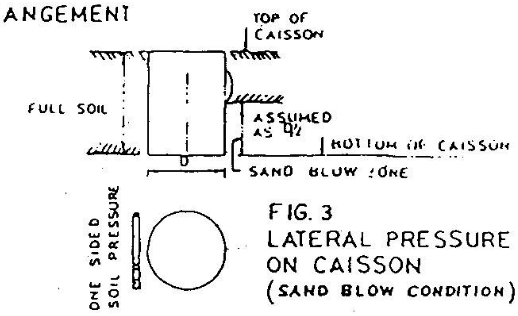
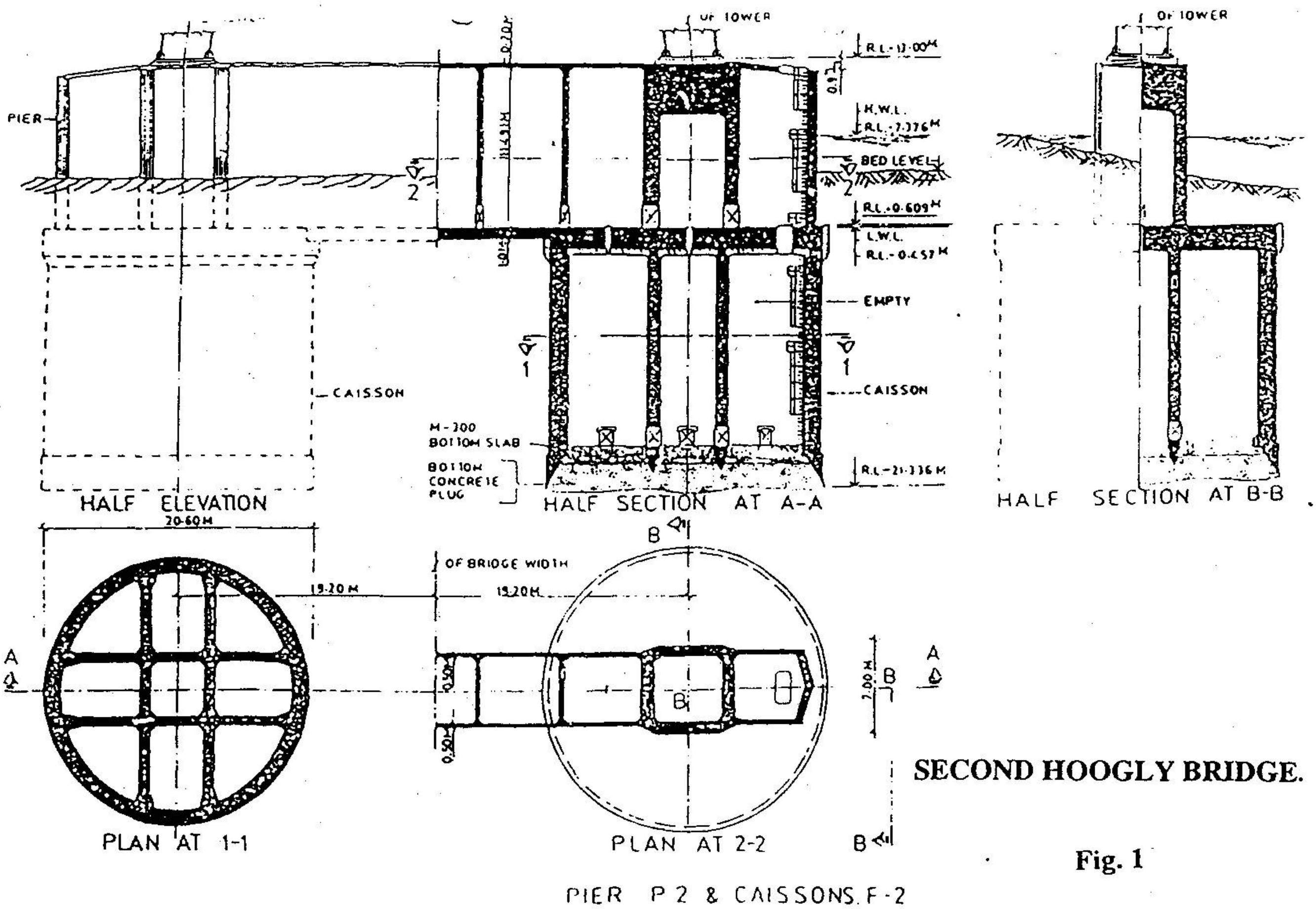
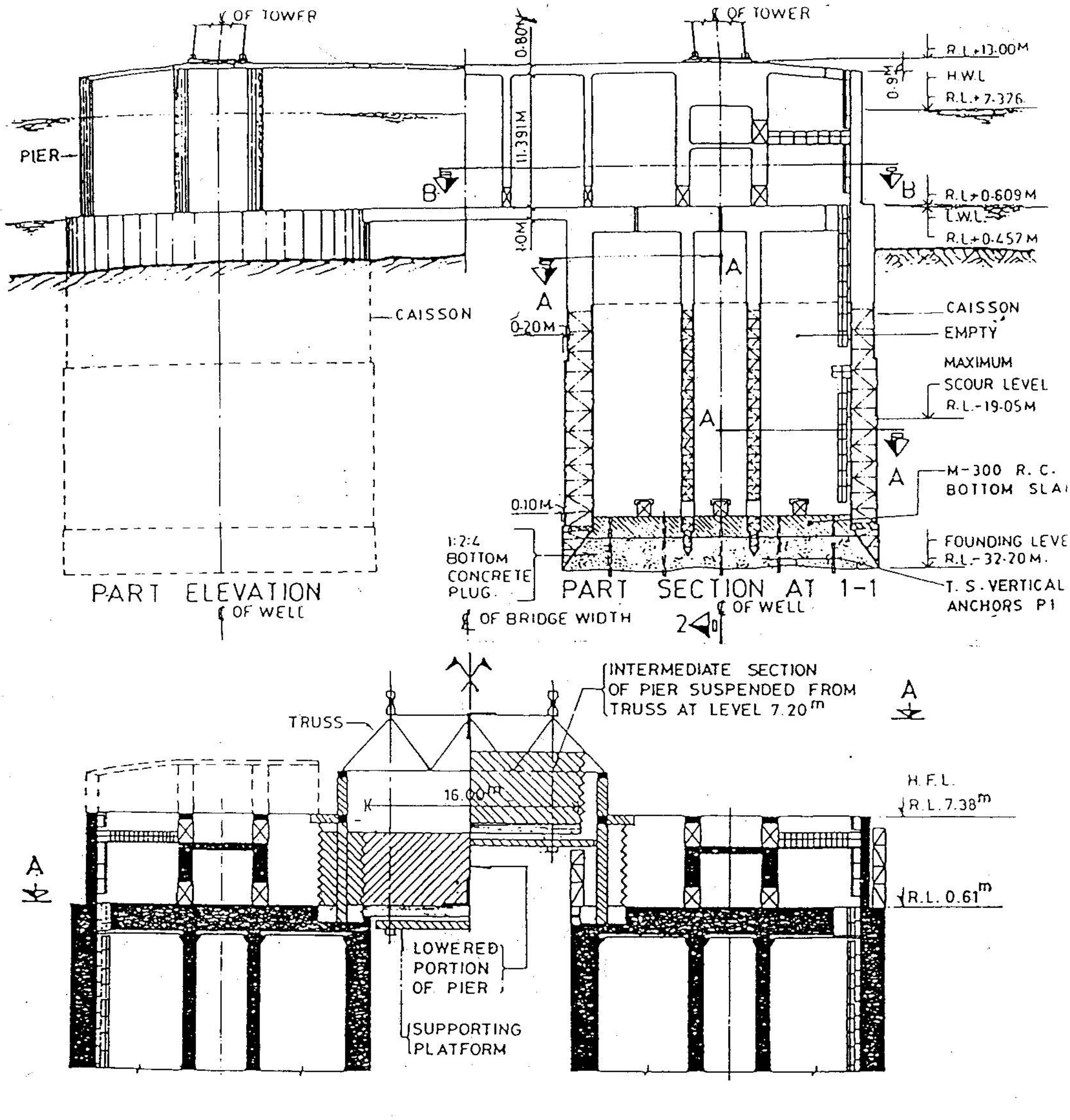


FIG. 3 LATERAL PRESSURE ON CAISSON (SAND BLOW CONDITION)

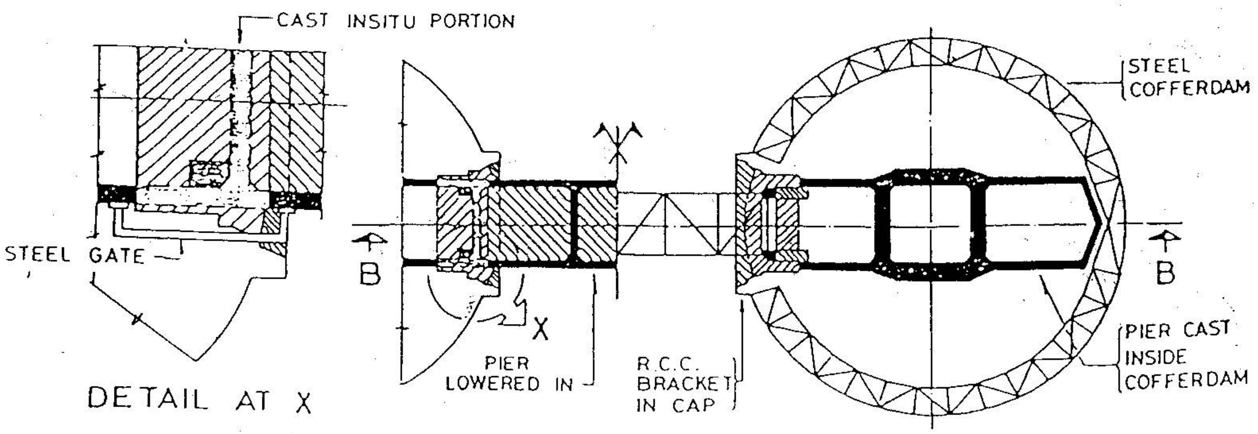


SECOND HOOGLY BRIDGE.

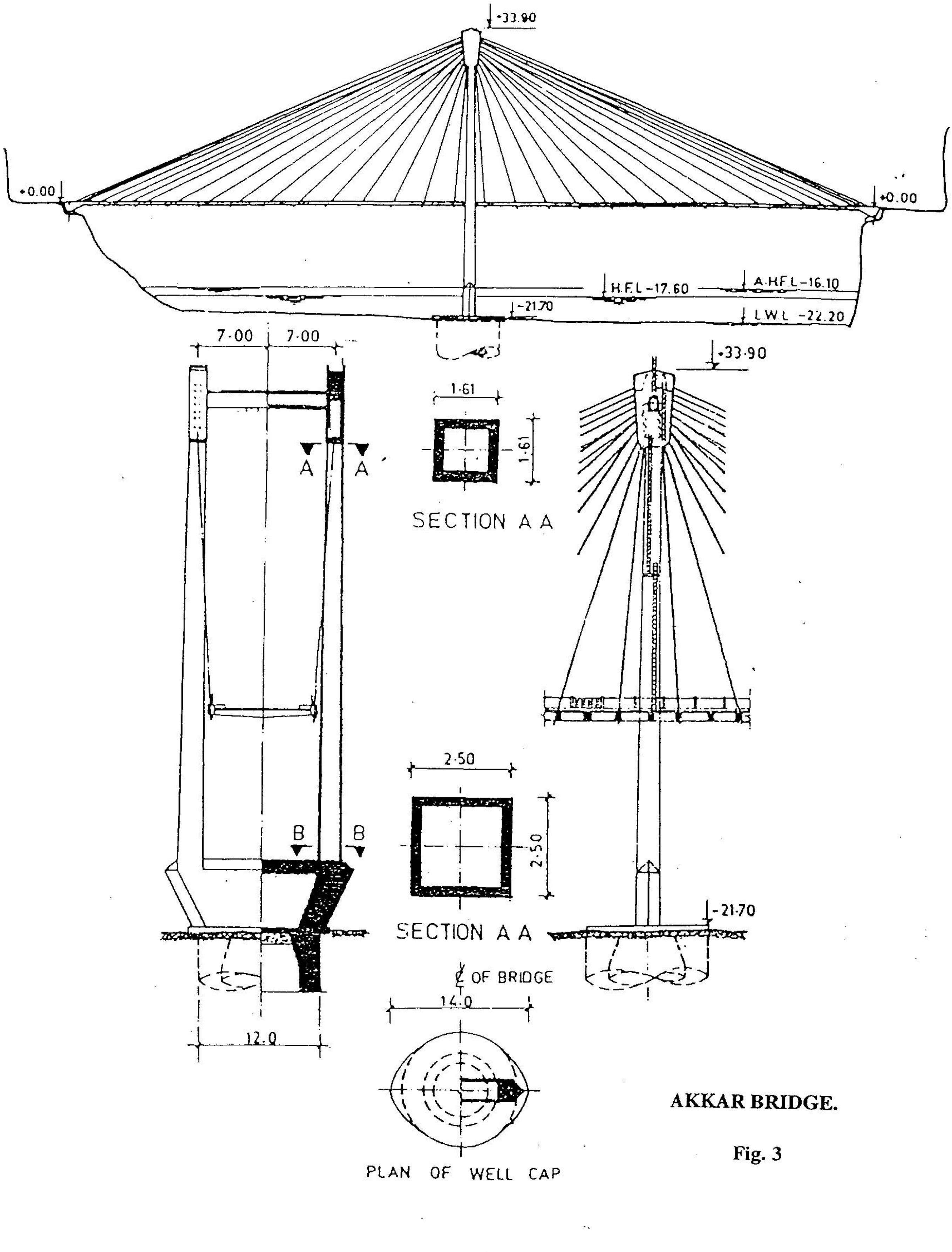
Fig. 1



PIER P3 ⊄ CAISSONS F-3

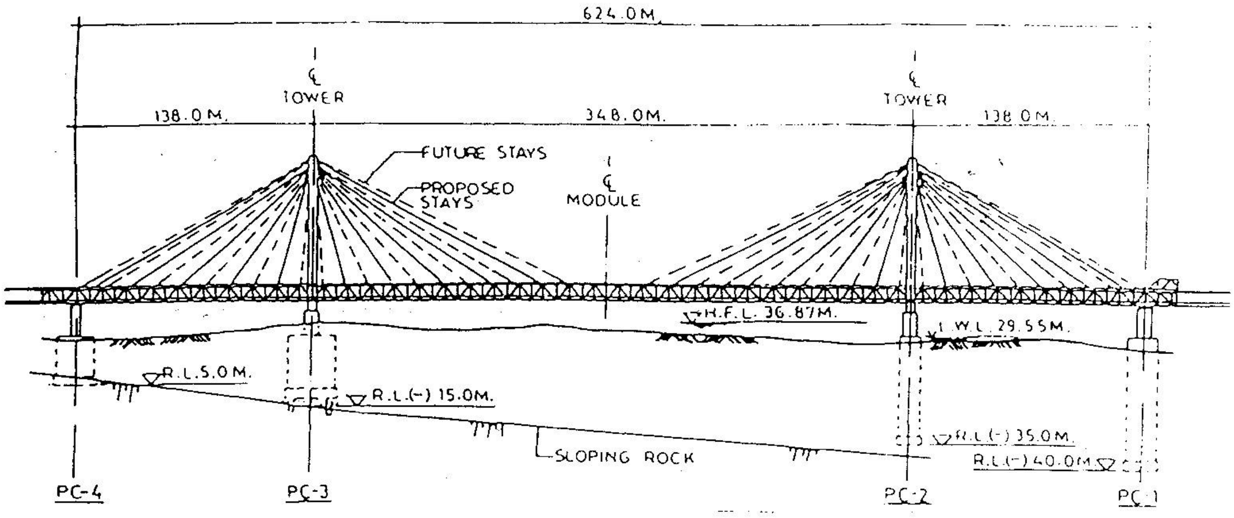


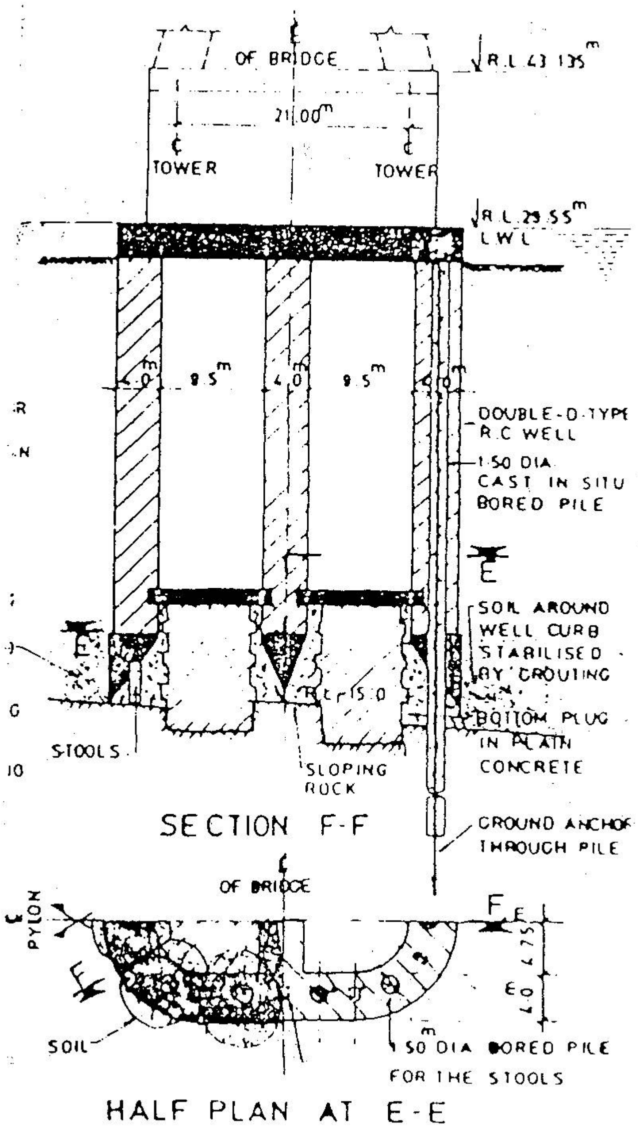
SECOND HOOGLY BRIDGE. Fig. 2



AKKAR BRIDGE.

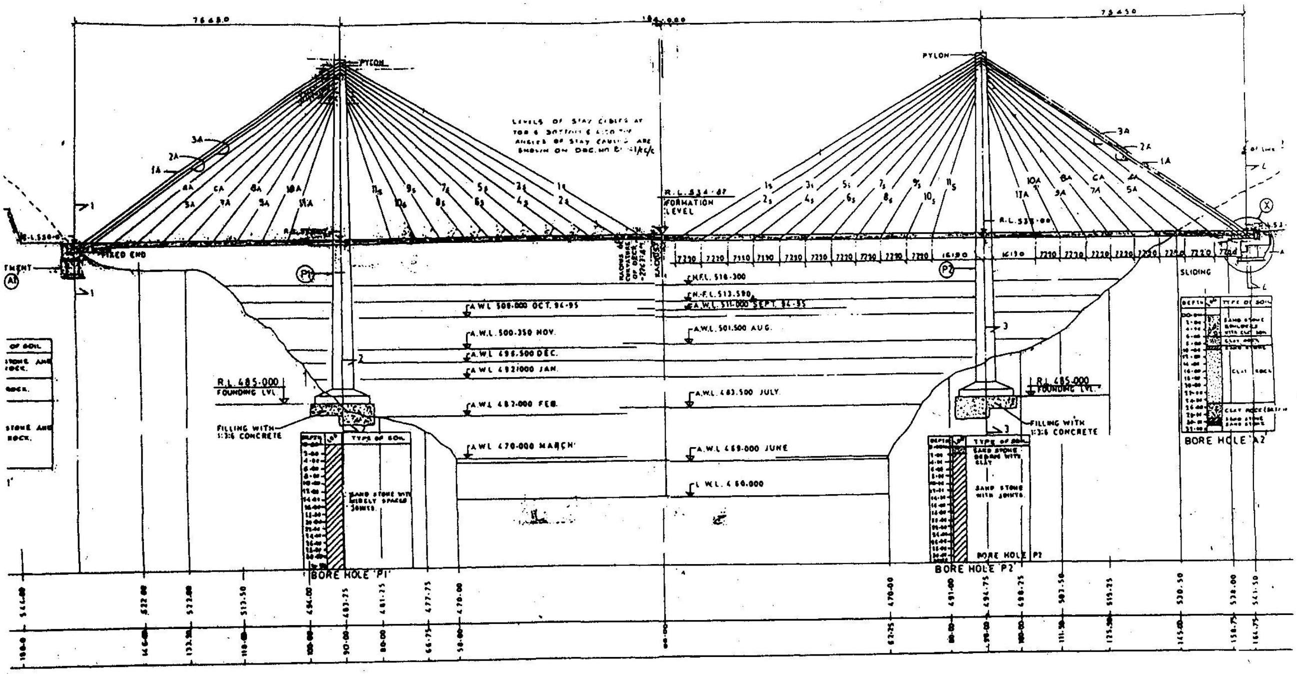
Fig. 3



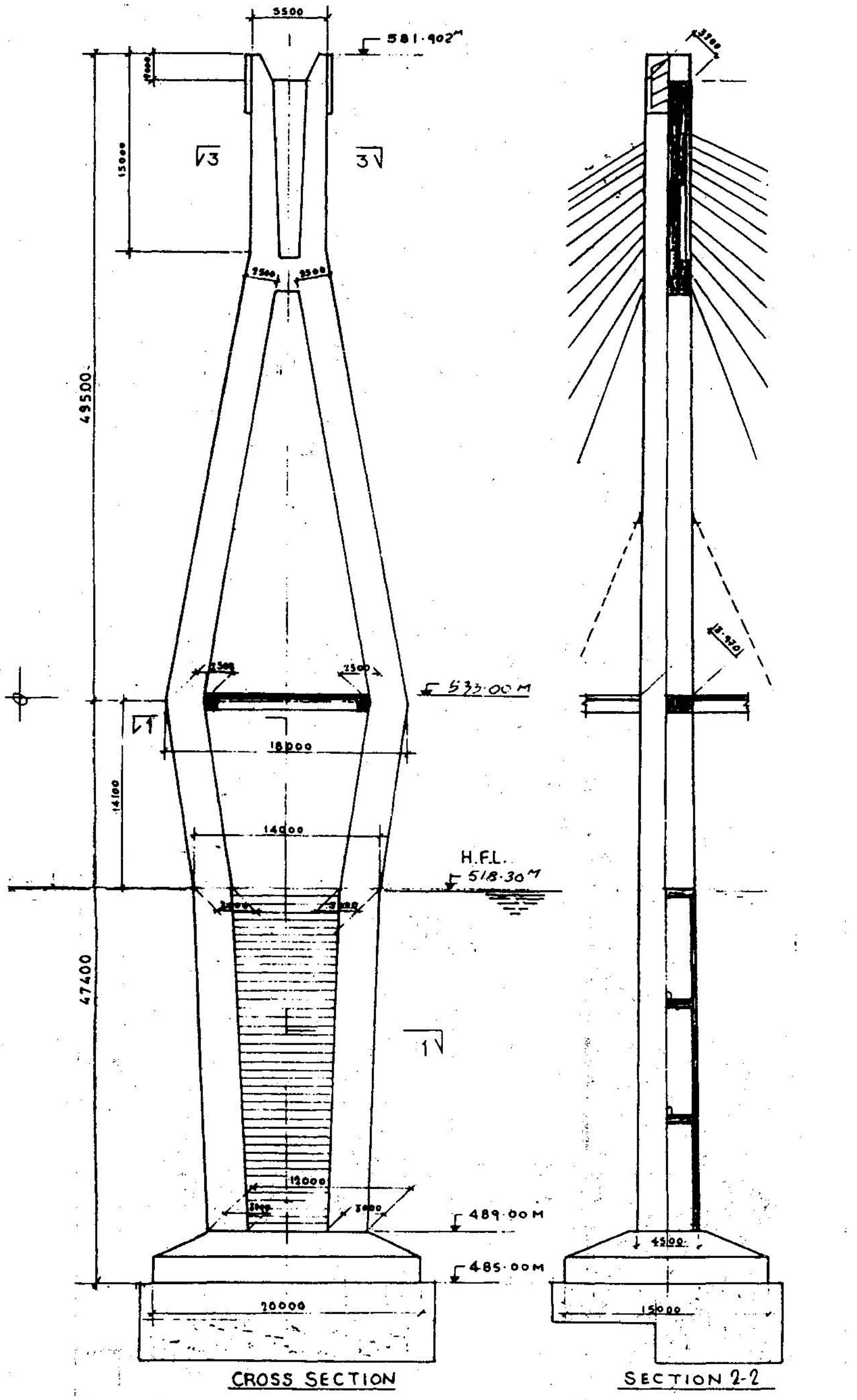


JOGIGOPA BRIDGE.

Fig. 4



BAGCHHAL BRIDGE **Fig. 5**



DETAILS OF PYLON

BAGCHHAL BRIDGE **Fig. 6**