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Design and Construction of Foundations for Akashi Kaikyo Bridge

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ABSTRUCT

The Akashi Kaikyo Bridge, the longest suspension bridge as ever, is just completed in 1998 as part of the Honshu-Shikoku Bridge project. This bridge has a main span length of 1991m and a total length of 3991m, and crosses over the Akashi Strait which is 4km in width and 110m in maximum depth on the bridging route. The maximum tidal current in the strait is as high as 4m/sec.

This paper describes outline of Honshu-Shikoku Bridge Project and some technical features of Akashi Kaikyo Bridge's foundation as follows.

- All foundations were designed against severe earthquake with a newly established seismic design method.
- Big and deep foundations for the anchorages were constructed with various new technologies.
- The main piers were constructed as spread foundations by Laying-down caisson method under the condition of deep sea and rapid tidal current.
- Newly developed low heat type cement and concrete of various mixtures were used.

1. INTRODUCTION

The purpose of the ongoing Honshu-Shikoku Bridge (HSB) Project is to link the island of Honshu and Shikoku via three routes, and thereby promote balanced regional development in Japan. The Honshu-Shikoku Bridge Authority was then founded in 1970 based on a law as an execution body of the project. Fig.-1 shows a concept and bridges of the project.

The Kobe-Naruto Route, the eastern route of the HSB Links, provides an 89 km, 6-lane (partially 4-lane) highway with a design speed of 100km/hr, and is just completed in 1998. This route contains two long span suspension bridges: the Akashi-Kaikyo Bridge and the Ohnaruto Bridge.

The Kojima-Sakaide Route, the central route of the HSB Links, features a 37km, 4-lane highway with a design speed of 100km/hr and a 2-track railway for ordinary trains. This route was opened to traffic in 1988 as the first direct link between Honshu and Shikoku. It contains six long span bridges over the 9.4km strait, which are collectively known as the Seto-Ohashi Bridges.

The Onomichi-Imabari Route, which includes ten long span bridges linking nine relatively large islands, is a 60km. Construction work on this route will be completed in the spring of 1999.

The foundations for Honshu-Shikoku Bridges are mainly located on the shore and the rest in underwater and generally bedded on granite or Izumi Formation layer, which is hard and old rock before the Oligocene or Cretaceous.

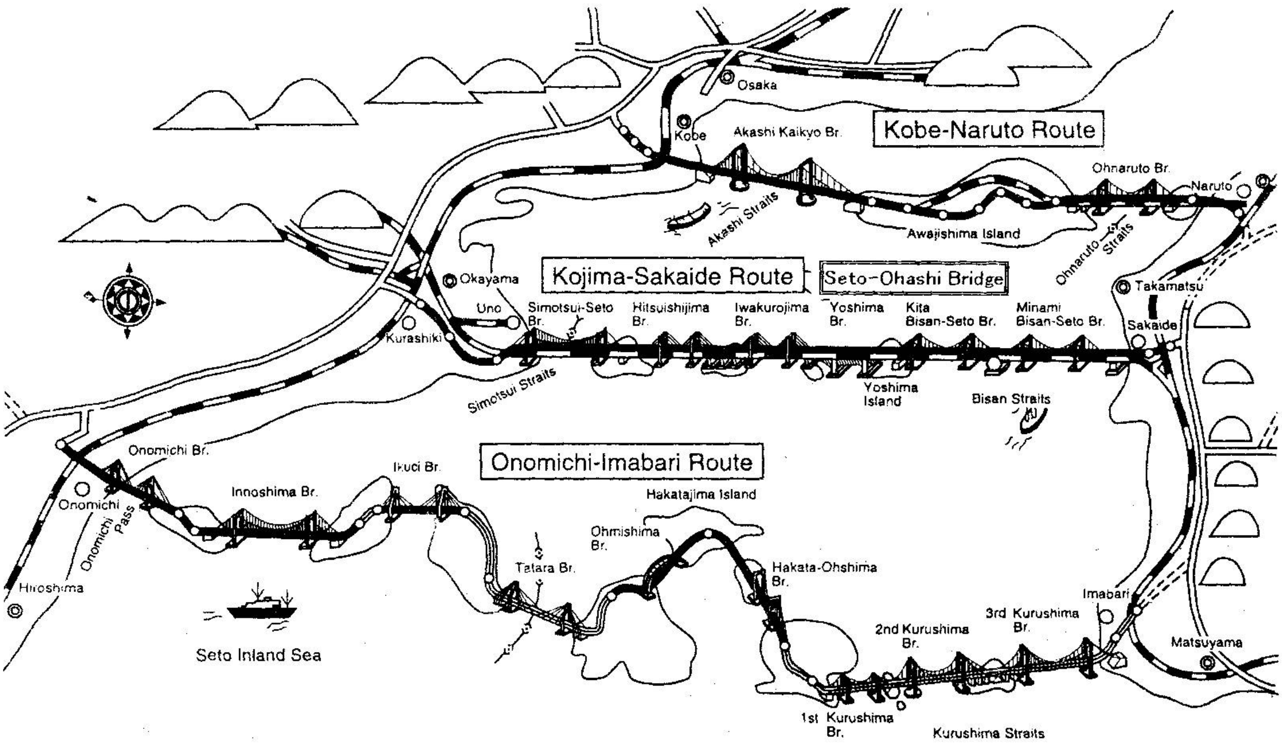

Fig.-1 Bird's Eye View of Honshu-Shikoku Bridges

2. AKASHI KAIKYO BRIDGE

2.1 Outline of the bridge

The Akashi-Kaikyo Bridge, crossing over 4km wide Akashi Strait, is a strengthened steel truss suspension structure with 3 spans and 2 hinges at 2 main piers. A total length of the bridge is 3911m and length between the 2 main piers is 1991m and fraction in each length is due to the Hyogo-ken Nanbu Earthquake. The bridge is supported by 2 anchorages (1A and 4A) and 2 piers (2P and 3P) on direct foundations, as shown in Fig-2.

The foundations of Honshu-Shikoku Bridge are generally bedded on hard rock layer mentioned above. As the foundations of Akashi-Kaikyo Bridge except for that of 4A, however, the granite bed is so deep, it was impractical to form on granite bed, eventually had to construct on relatively soft and young rock.

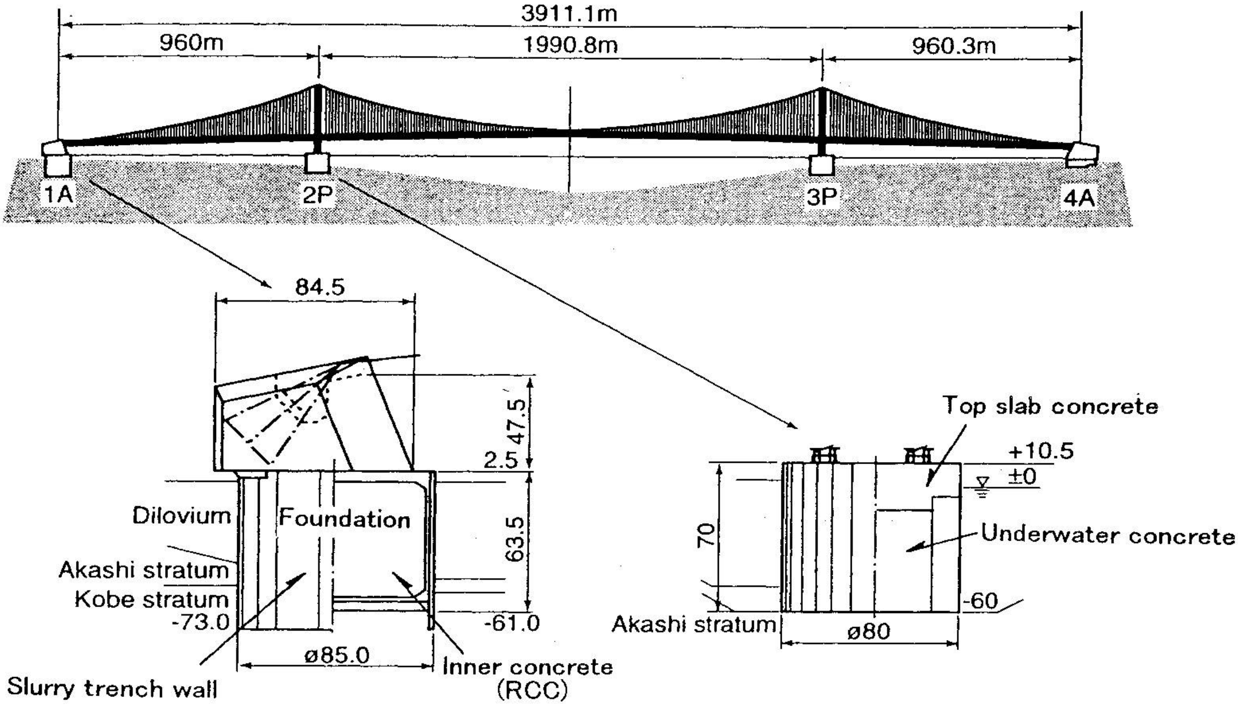

Fig.-2 Profile of Akashi-Kaikyo Bridge

2.2 Design condition

2.2.1 The Akashi Strait

The Akashi Strait is designed as a statutory waterway with width of 1500m where approximately 1400 ships pass daily.

The maximum water depth in the central part of the strait reaches approximately 110m.

The maximum current speed is approximately 3.5m/sec at 2P and 4m/sec at 3P. An observation near the site showed that mean wave height is 51.4cm and the maximum wave height is approximately 6m during a typhoon. Tidal level change is observed to be approximately 1m.

2.2.2 Geological condition

The seabed consists of recent and upper Pleistocene deposits, Akashi Formation of the Pleistocene to Pliocene, Kobe Formation of the Miocene and granite as shown in Fig-3. The Akashi Formation is composed of semi-cemented gravel and sand, and contains a lot of decayed gravel. The Kobe Formation is made of weakly cemented soft rock with complicated alteration of thin sandstone and mud-stone layers. Namely, 1A and 3P are founded on the Kobe Formation, and 2P is put on the Akashi Formation. On the other hand, 4A is designed to be rested on weathered granite.

3. DESIGN OF FOUNDATIONS FOR AKASHI KAIKYO BRIDGE

3.1 Soil investigation

In order to obtain the engineering properties of the subsoil for the foundation design, geotechnical investigations were carried out in several phases. They included geophysical survey, exploratory drilling, laboratory soil and rock tests, seismic activity investigation, etc. Immediately before the construction started, a detailed investigation was carried out to finalize the foundation design.

The detailed investigation had the following features.

- Large-size self elevated platforms (SEP) were used to conduct exploratory drilling in deep water with rapid current.
- Undisturbed samples were collected continuously from various types of rock including 300mm diameter continuous sampling from semi-cemented gravelly sand layer of Akashi Formation.
- Engineering properties of rocks were determined using various types of in-situ tests such as geophysical logging, pressure-meter tests and permeability tests.

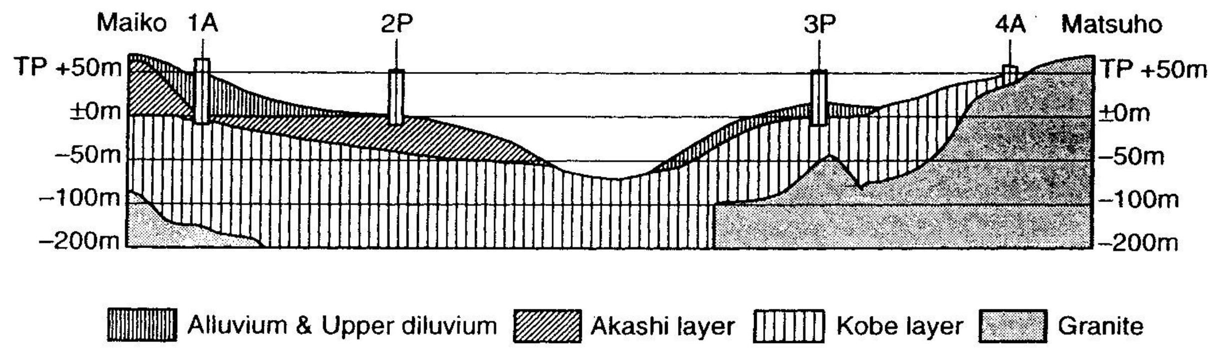

Fig-3 Geological Section of the Akashi Strait

3.2 Seismic Design

Conventional seismic design standard for Honshu-Shikoku Bridges is mainly applicable to the foundations supported with hard and old rock layers. However, Akashi-Kaikyo Bridge in which huge foundations was to be constructed upon softer and younger layers, a new seismic design standard had to be formed, because non-linearity of the bearing layer as well as dynamic interaction between the ground and foundation were judged to be not negligible.

The concepts of the new seismic design standard for the Akashi-Kaikyo Bridge's foundations are as follows.

- The response of the foundations is to be obtained through the response spectrum method with corresponding 2-degree of freedom system, which models both rigid foundation and ground springs to express the oscillation in rocking mode as well as swaying mode.
- The modes should be established with consideration of dynamic interaction between the foundation and the ground (see Fig.-4).
- In order to evaluate the dynamic interaction, local ground condition around the foundations should be precisely taken into account.

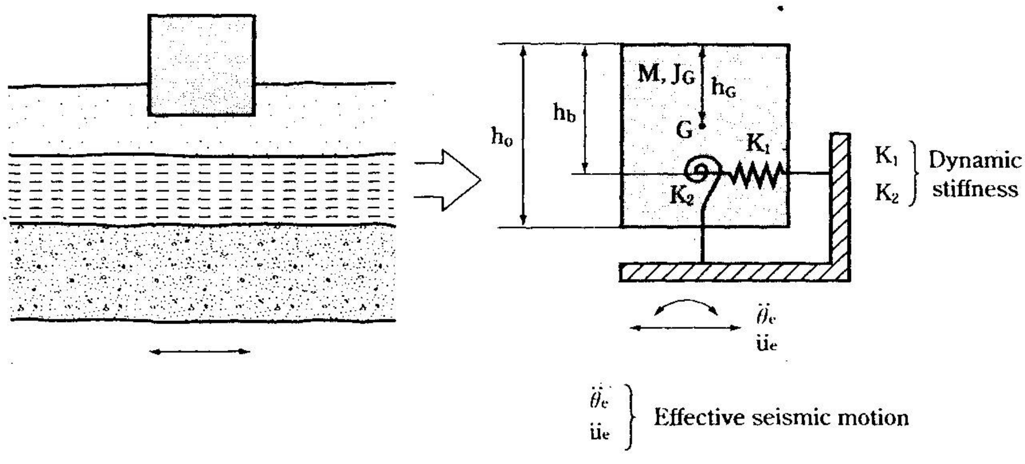

Fig-4 Dynamic response analysis model

4. CONSTRUCTION OF FOUNDATIONS FOR AKASHI KAIKYO BRIDGE

4.1 Deep-water underground slurry wall

The bedrock which could serve as the supporting ground for the 1A anchorage is approximately 60m below sea level. After repeated consideration, a method that may be called "deep-water underground slurry wall method" was employed. In this method, retaining walls arranged in circular form were installed first, and the soil inside these retaining walls was excavated in the open-air while the ground water inside was pumped out. A continuous underground wall with 92 sections of the same length was constructed using an excavator for continuous wall construction that was one of the largest in Japan. Using this as a retaining wall, the 84m diameter inside ground was excavated. The excavation work was started at 2.5m above sea level and reached about 61m below, taking about 11 months in total to complete, during which approximately 330,000m^3 of soil was excavated. After the excavation, RCC (Roller Compacted Concrete) was applied to make a foundation consolidated with the retaining wall.

4.2 Laying-down caisson method

Construction method for the foundation of the two main piers (2P and 3P) has been named "Laying-down caisson method" and used in many bridges in the Honshu-Shikoku Bridges including Seto-Ohashi Bridges. Although it is called "caisson", a foundation constructed by this method is not, in fact, a caisson (a rigid foundation which has an effective embedment), but a spread foundation rested on the pre-excavated bearing layer without any embedment.

The main characteristics of this method are as follows.

- Excavation up to the sufficient bearing layer and the building a caisson (a steel form for underwater concrete) are split into separate procedure.
- As a result of this separation, the method of excavation and machinery for it are not restricted from the inner size of the caisson. It is thus possible to shorten a time for excavation.
- After completion of the excavation, the caisson is towed to the site as shown in Photo-1, sunk with high accuracy to the position specified in the design, whereupon underwater concrete is cast into the bulk of the foundation as rapidly as possible, and ordinary reinforced concrete is applied to the top part so as to complete the entire main pier.

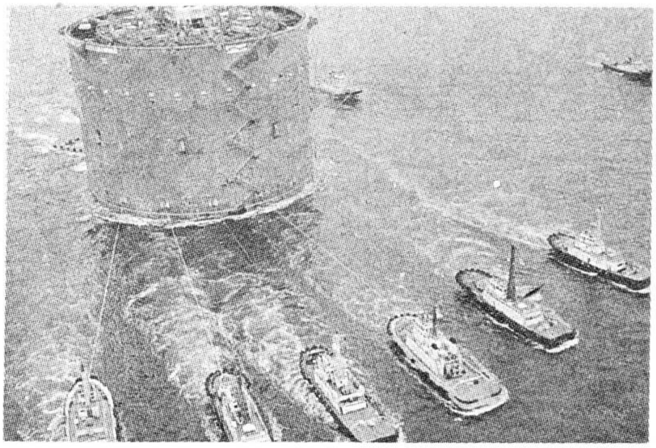

Photo-1 Laying-down Caisson Method (Towing Steel Caisson of 2P)

4.3 Concrete material

4.3.1 Low-heat cement

The cement previously used for mass concrete in the Honshu-Shikoku Bridges was either portland blast furnace slag cement or low heat type blast furnace slag cement.

However, occurrence of thermal cracking could not be always suppressed within satisfactory level, because the concrete was rather rich mix with 280kgf/m^3 (2740N). The adiabatic temperature rise of this concrete was measured to be about 45℃.

It was thus decided to develop a low heat cement for the Akashi-Kaikyo Bridge, by which the concrete with the unit content of 260kgf/m^3 (2540N) can be mixed so as to have the adiabatic temperature rise of less than 25℃. This cement utilizes pulverized blast furnace slag, whose content can be enhanced to 80% at maximum, and fly ash is sometimes added to adjust component design of cement. This newly developed cement was widely used in various part of the substructure of the Akashi-Kaikyo Bridge.

4.3.2 High fluidity concrete

For the concrete work of the anchorage, a large volume of concrete was required to cast into the space congested by re-bars and structural steel, within the limited construction period. Large amount of workers and equipments are necessary by the conventional methods. Hence, the newly developed highly fluidity (self-compactable) concrete was adopted for the concrete of the anchorage. It is a mixture of low-heat cement with an admixture of limestone powder, adding air entraining and high-range water reducing agents. The fluidity is controlled by a slump-flow, prescribed within 45cm to 60cm. Due to the high fluidity of the concrete, it flows into the very complicated space without any vibrators operated by workers. The adoption of the high fluidity concrete results in the reduction of workers and construction period, without losing the quality of the concrete.

5. CONCLUDING REMARKS

The important technical aspects in the design and construction of foundations for the Akashi-Kaikyo Bridge are briefly described in this paper. Many other technologies, not described in this paper, were developed and actually applied to the construction.

The authors believe that all the technologies, developed and applied in the Akashi-Kaikyo Bridge, can contribute to the accomplishment of ultra-long span bridge projects in the future.

Innovative Structural Solutions and Construction Techniques for Deep Foundations of Large Bridges Over Rivers.

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SUMMARY
This paper deals with the development of new structural solutions adopted for deep foundations of bridges over the large rivers Volga and Kama. The innovative structural solutions called for new efficient construction techniques which were based on the use of rising floating platforms. Also some typical practice of quality control of downhole enlargement and concrete strength of bored piles are discussed.

1. INTRODUCTION

The design and construction of large bridge crossings over the rivers and reservoirs in the basin of the Volga river have their own specifics. Specific geological and hydrological conditions are characterized by low air temperature, reaching minus 35°C; intensive action of ice drift, having a thickness of up to 1 m; water depths of up to 30 m; a significant variability in water levels, a river bottom composed of sands susceptible to scours, and underlying by clays of low bearing capacity. By experience the construction of piers in such complicated conditions takes about 70% of labor intensity and time compared to that of the whole bridge and typically 60% of overall bridge cost. To improve this situation a new structural solution and construction techniques have been developed.

2. NEW STRUCTURAL SOLUTION AND CONSTRUCTION TECHNIQUES

2.1 Structural solution details

Compared to the traditional single pilework option, a new structural solution comprises two separate pilecaps. This resulted in a reduction of ice effect on foundation. Raising a base of pile cap above working water level and adoption of ice protective shell allowed to avoid construction of sheet piling and subaqueous concreting.

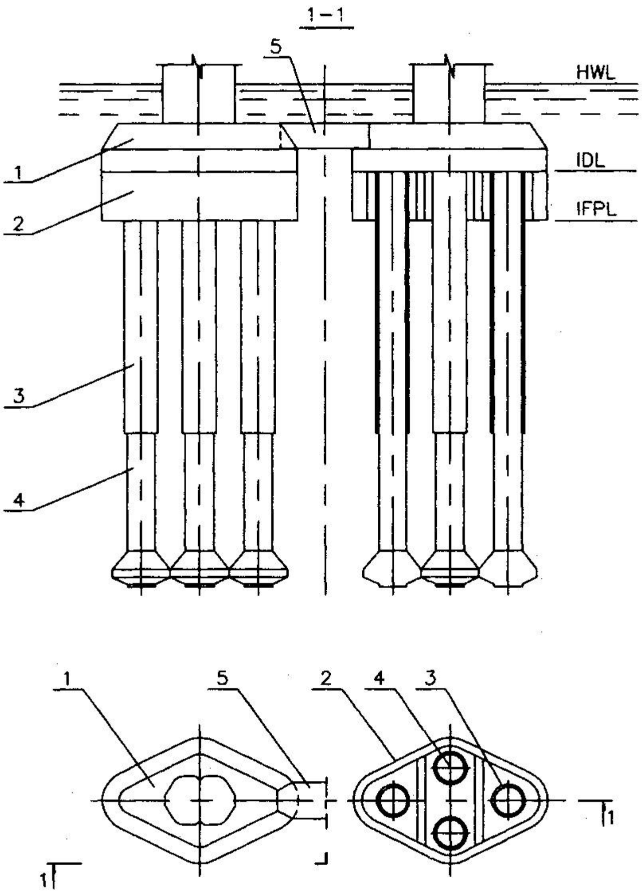

Fig. 1 General arrangement of foundation: 1 – pile cap, 2 – ice protective shell; 3 – steel encasement; 4 – bored pile; 5 – strut; HWL – high water level; IDL – ice drift level; IFPL – ice first push level (min)

At the same time the pile cap is located within the water level of possible ice floating and therefore the rhombus, streamlined shape is given to the cap. The ice protective shell repeats a form of pile cap. The purpose of this structural detail is to eliminate some concrete volume (to reduce the pier weight) and to protect against floating of ice. Special structural arrangements in the foundation with shell do not allow formation of ice inside of it. Positive temperatures inside the shell are maintained for a long period of time. Therefore a number of freezing/thawing cycles for concrete of piles is reduced. Another additional purpose of this cover is for aesthetic appearance of pier at low water levels.

Typical pier foundations comprise reinforced concrete bored piles of 1.5m in dia penetrated to a depth of up to 50 m protected by a steel casing of 2.0 m in dia within the probable depth of scour, and 1.7 m in dia at a lower part between the scour level and the level of bearing stratum. To increase the bearing capacity and reduce a number of piles in foundation, the enlarged up to 3.5 m pile base is normally adopted. In the recent bridge projects, where bridges were designed with spans of up to 160 m, normally four bored piles of large diameter (2m dia typical) were adopted for each pilework. A general arrangement of pier foundation adopted for the bridge over the Volga river near Saratov is shown in Fig. 1. To reduce a magnitude of pier top displacement due to the ice loads, a strut between pilecaps have been designed. This element provides for load distribution between upstream and downstream pilecaps.

Another design aspect is that steel pipes (casings) is designed as contributing to the resistance of bored piles. Thus bored piles act as a combined section. This approach increased design characteristics of piles and allowed to reduce a quantity of reinforcement within the casing length, and provided an improved quality for concrete laying.

2.2 Construction techniques

Construction of deep foundations is based on the use of floating platforms PMK and techniques of installation of bored piles with casings and enlarged to a 3.5m base. The floating platforms are formed of pontoons and have a stiff bearing on river or reservoir bottom by means of lowered posts (columns) which are connected to the platforms (Fig. 2). Placement of bored piles is conducted from top of floating platforms. First, the columns are lowered to a river bottom and driven in soil using a vibrator to a depth ensuring stability of platforms. Installation of steel casings (typically of 2.02 m in diameter) is implemented by the vibrator from operating bridge installed over the platform. To construct bored piles with enlarged bases a boring rig "Kato-50" is adopted which is placed over the platform.

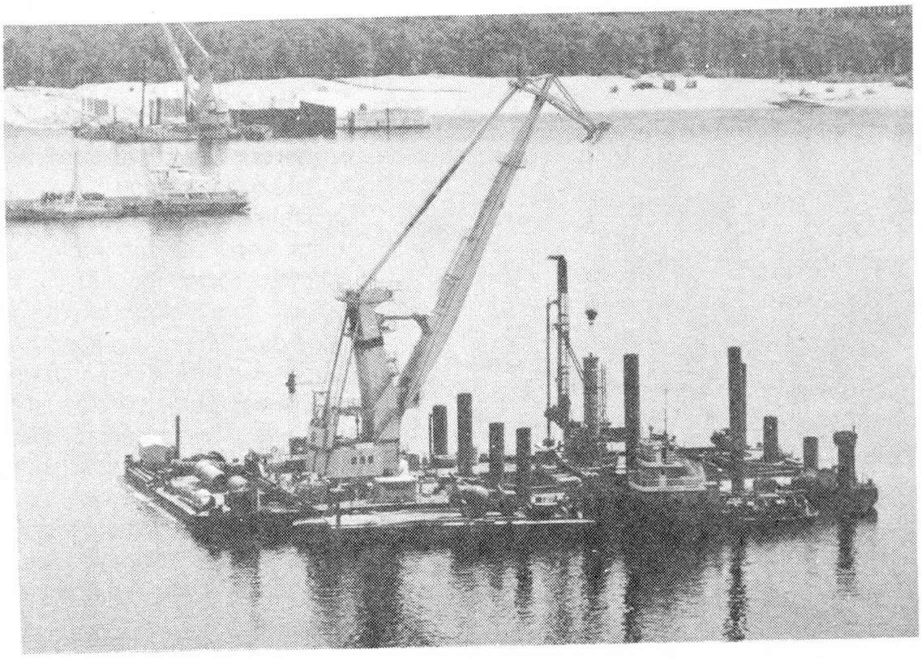

Fig. 2 Typical floating platform

Pilework comprises two elements – pile cap and ice protective shell which are constructed in dry conditions above working water level. Construction of ice protective shell is conducted above the water over a bearing mould fixed on top of steel casings. The sequence of operations is as follows: 1 - two precast reinforced concrete diaphragms of the shell are installed at the mould, 2 - external steel forms are placed to form a circle section, 3 - reinforcement of the shell is placed, 4 - internal steel forms are installed, 5 - ice protective shell is concreted and when the concrete is hardened, the forms are removed. Final configuration of ice protective shell constructed for the bridge over the Volga river near Saratov is shown in Fig. 3.

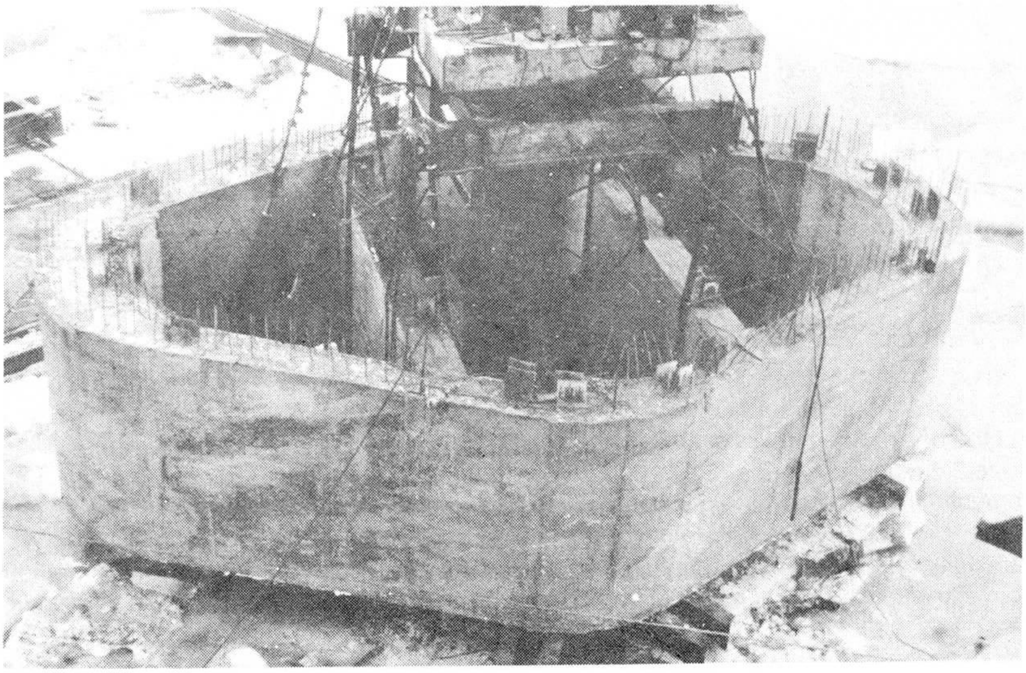

Fig. 3. Ice protective shell

To lower the ice protective shell, typically having a mass of 240 t, into its final position, a special builder's lift have been adopted. This builder's lift consists of bearing frame, fixed on top of casings and a post fabricated of the column of floating platform (Fig. 4). Columns are embraced by ties, between which electro jacks are placed. When the shell is lowered to a final elevation by means of these jacks, the builder's lift is dismantled.

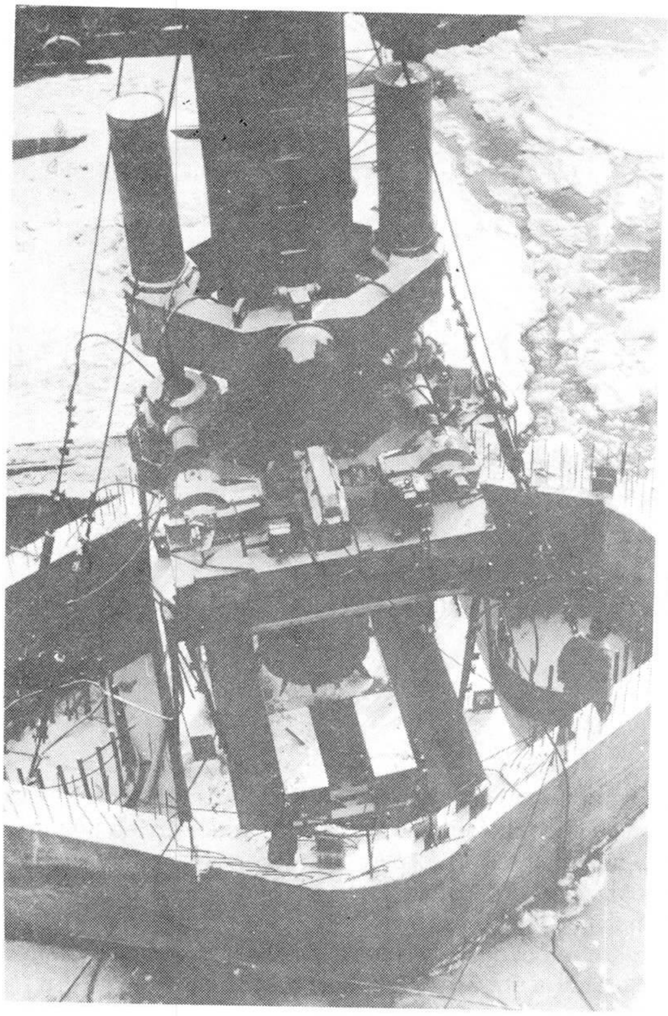

Fig. 4 Builder's lift used for lowering ice protective shell

Final position of ice protective shell is geodetically controlled in plan and elevation. Internal void of shell is spanned by scaffolding formed of concrete plates fixed by embeds on top of shell. These precast plates repeat the shape of internal contour of shell. Reinforcement of pile cap is placed in two stages. At the first stage, steel vertical panels of forms for lower pile cap portion are installed over erected mould and connected by bolts into a single circle section. Then reinforcement is placed in this pile cap portion. After that panel forms for top pile cap portion are installed and placement of reinforcement is completed. Concreting of pile cap is implemented continuously by inclined layers. To prevent thermal losses, moisture protective mats is normally used for covering a surface of pile cap and panel forms.

5. QUALITY CONTROL

The quality of construction of pile foundations is thoroughly controlled. It includes, but is not limited to, checking straightness, tilt of the borehole and enlargement shape if applicable, determining the continuity and strength of concrete in bored piles, confirming the strength of bearing stratum (load tests, etc). Some specific areas are discussed below.

To control a design shape, dimensions and concentricity of downhole enlargement of the bored piles, a special control system has been developed. Lowered into a borehole device provides information on shape, dimensions and volume of the enlargement to a computer. A typical computer graph, showing measurement results of borehole enlarged to 3.5 m is given in Fig. 5. If required the enlargement may be adjusted to a design configuration and measurements are repeated. When satisfactory results are achieved, piles are allowed to be concreted.

To control continuity and strength of concrete in bored piles, an ultrasonic device is used. Control measurements are normally implemented at one pile of each foundation. Channels are arranged in a controlled bored pile using steel pipes. A typical scheme of channels location for ultrasonic control in a bored pile is shown in Fig. 6. Results of control are presented in a form of table-graph. An illustration of measurement results, showing values of concrete strength at one of the bored piles at pier # 18 for the bridge crossing over the Volga river near Volgograd, is given in Fig. 7.

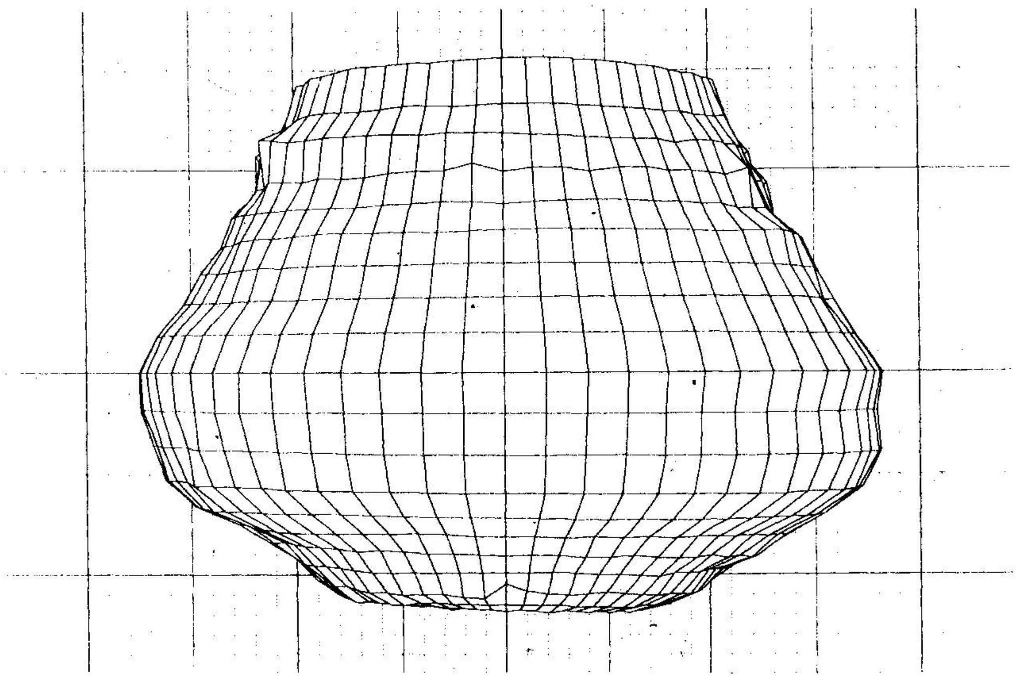

Fig. 5 Graph showing dimensions and shape of borehole enlargement

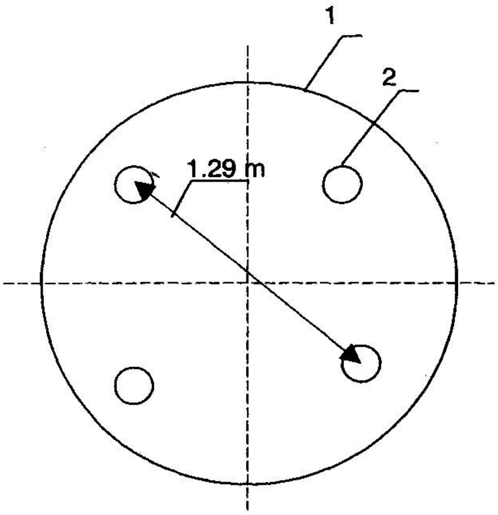

Fig. 6 Scheme of channels for determination of concrete strength in bored pile: 1 – bored pile, 2 – channels (coreholes exaggerated)

Distance from top to point of measurement, m	Device readings at base of 1.29 m	Device readings adjusted to base of 1.0 m	Standard strength B30 (240)
0.5	328	254	
1.5	326	253	
2.5	315	244	
3.5	312	242	
...	...	...	
11.5	324	251	
12.5	325	252	
13.5	325	252	
14.5	320	248	
...	...	...	
25.5	342	265	
26.5	336	260	
27.5	318	247	

Fig. 7 Table-graph resulted from ultrasonic control of bored pile

6. CONCLUSION

The abovediscussed innovative structural solutions and construction techniques have been implemented for a number of bridge foundations and proved their efficiency in practice. Bridge projects currently under construction are as follows. The bridge over the Volga near Saratov has a length of about 2.2 km (main span of 157.5 m) and allows traffic of two lanes in each direction. The bridge over the Volga river in Volgograd has a length of about 1.2 km (main span of 155 m) and allows traffic of three lanes in each direction. The bridge over the Kama river is of about 1.6 km in length, having a main span of 150.5 m, and accomodates traffic of two lanes in each direction.

Thirty pier foundations for the bridge over the Volga river near Saratov have recently been built. A final configuration of piers constructed for this bridge is shown in Fig. 8. The similar type of pier foundations for the bridges over the Volga river near Volgograd and over the Kama river near Kazan are currently under construction.

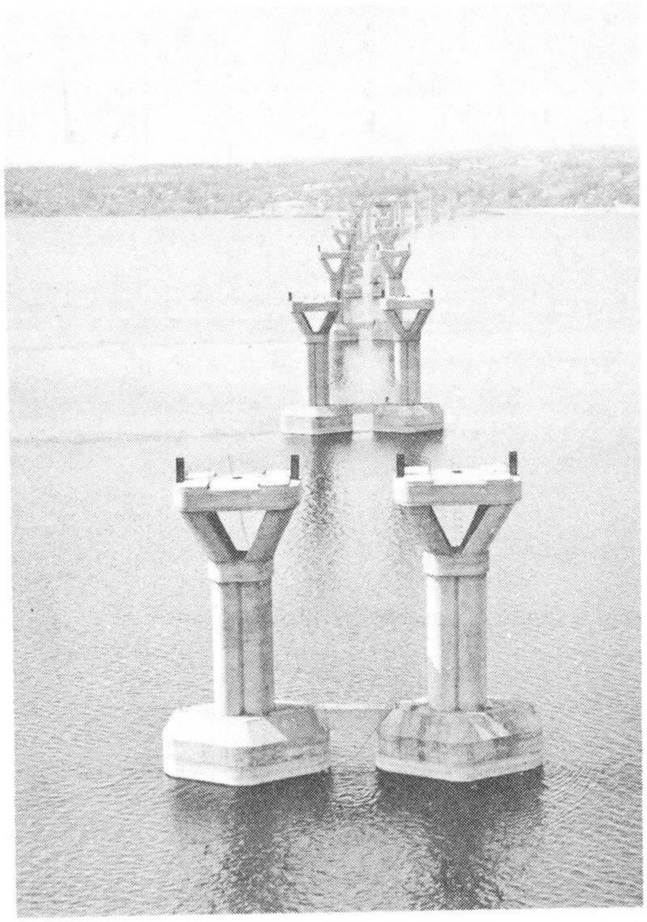

Fig. 8 View of constructed piers for the bridge over the Volga river near Saratov

Special Method of Well Sinking Adopted at New Nizamuddin Bridge on National Highway-24 in New Delhi

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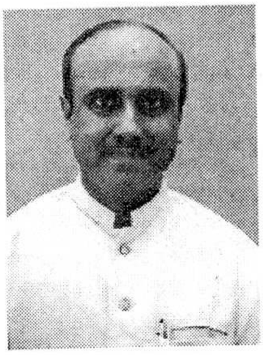

Born on the 21st July, 1945, graduated in Civil Engineering from M.B.M. Engineering College, Jodhpur in 1968. He started his career with Delhi Municipal Corporation. Later, he joined the International Airports Authority of India. He is Founder Member and Secretary, Association of Airport Planners and Engineers (India). He was Vice-President, Indian Roads Congress, 1991-92, and again he was elected as its Vice-President for 1997-98. Recently, he has been elected as Vice-Chairman of Consultancy Development Centre, New Delhi.

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Born on 1st January 1939, did his Bachelor of Engineering (Civil) from University of Roorkee, India in 1961. Since almost the beginning of his career, he has been engaged in execution of a number of bridge projects both in India and abroad. On behalf of ICT as domestic consultants, he was responsible for execution of Second Nizamuddin Bridge in New Delhi.

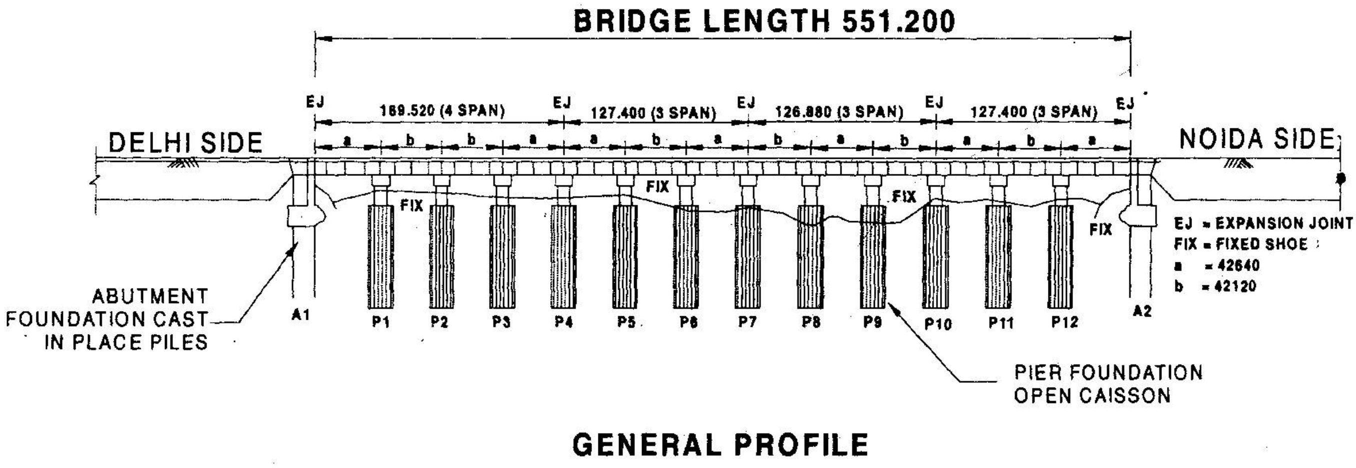
SUMMARY

The Paper deals with the special technique adopted for sinking of well foundations in the new Nizamuddin Bridge in Delhi. Sinking of wells by conventional method has long been adopted in construction of bridges in this country. In conventional method, the sinking is being done by dredging the sub-soil material from inside the well and the well would sink automatically by its own weight. Quite often, kentledge loads had to be applied on top of the steining to put extra weight for sinking along with dewatering from inside the well, water as well as air jetting along the periphery of the well steining. This was quite a slow and cumbersome process. The Japanese consultants and the contractors adopted a new technique in sinking of the well foundations, which has been adopted for the first time in this country. In this process, external load was applied on the well steining with the help of soil anchors and hydraulic jacks and simultaneous dredging of the material from inside the well. This process resulted in fast sinking of wells and this helped in saving of time and cost. This Paper describes mainly this specialised method of sinking.

1. INTRODUCTION

The New Nizamuddin bridge along with its short links on National Highway-24 over river Yamuna in New Delhi, the capital of India has been constructed under Japan Grant Aid scheme of the Government of Japan through Japan International Co-operation Agency (JICA). Detailed engineering of the project was carried out by M/s Nippon Koei Co., Ltd in association with Katahira Engineering International, Japan. They were also the supervision consultants of the project, Employer of the Project was Ministry of Surface Transport, Government of India. The work was supervised by Public Works Department, Government of Delhi on behalf of Ministry of Surface Transport. M/s ICT (Pvt.) Ltd. were the domestic supervision consultants with M/s Nippon Koei. The Project was executed by M/s Obayashi Corporation of Japan with M/s Engineering Construction Company (ECC) of M/s Larson & Tubro Group as their sub-contractors.

The bridge is 551.20 m long with 13 spans of 42.4 m (average) each c/c of bearings with short link approaches of 359.8 m on Delhi side and 419.2 m on Noida side. The width of the bridge is 22.6 m (end to end) consisting of 4 lane carriageway of 15 m with 3 m wide cycle track on either side. The superstructure is prestressed concrete precast multiple I girders (10 Nos per span) with RCC deck, having one unit of 169.52m of 4 spans with connection girder, one unit of 126.88 m and two units of 127.4 m of 3 spans each with connection girder. There are 12 Nos of wall type piers, 7.4 m to 8.3 m in height supported on RCC caissons. The abutments are 8 m high RCC wall type. The abutment foundations rest on cast-in place RC piles (18 numbers, 1 m diameter and 24 m deep). A general profile, and details of the proposed bridge (typical cross section) are shown in **Fig. 1.** The work was started in February 1996 and completed in Feb. 1998.



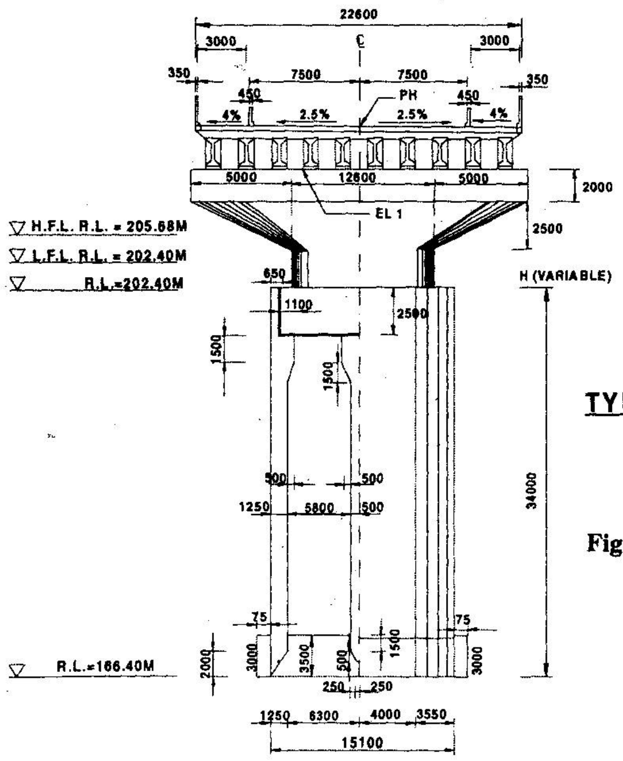
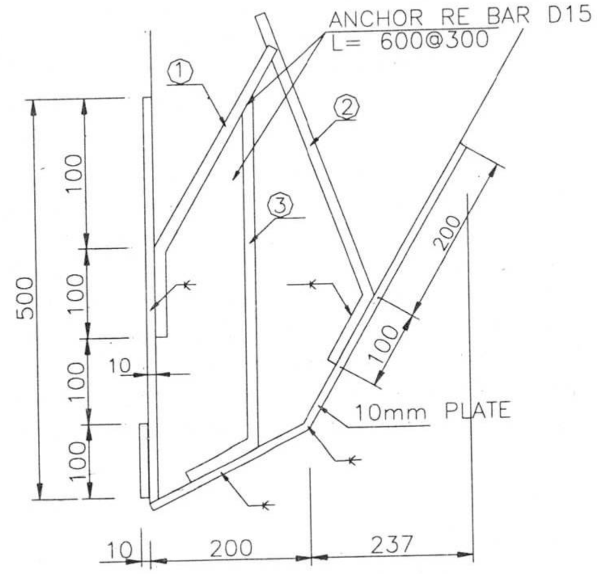


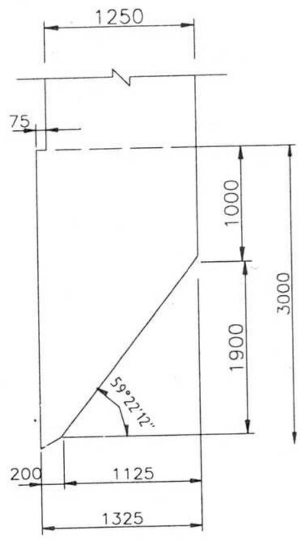
Fig.1 DETAILS OF THE BRIDGE

2. SIZE AND SHAPE OF CAISSON AND SUB SOIL STRATA

Oval shaped, double D type caisson foundations with overall size 15.10x7.10 m having 1.25 m thick RCC steining and a central diaphagrm of 1 m width in M-30 concrete have been provided. Caissons are 34 m deep below top of well cap. Well curb is 3 m high with 75 mm projection on outer face of steining. The inner face of curb has an inclination of 59° 22' 12". Cutting edge provided below the curb is made of steel plate as per shape shown in **Fig. 2**. **Photo 1** shows cutting edge in position and **Photo 2** shows sinking of caisson in progress.



Details of cutting edge



Details of curb

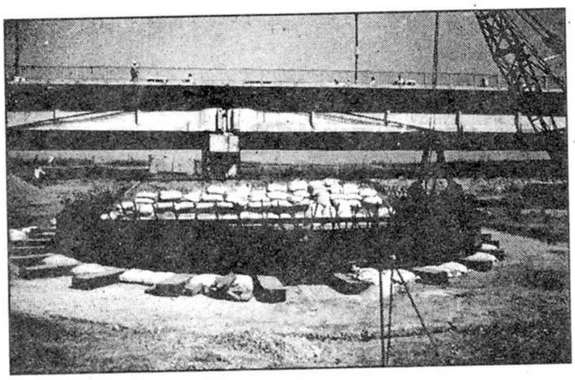


Photo 1 : A close view of cutting edge in position

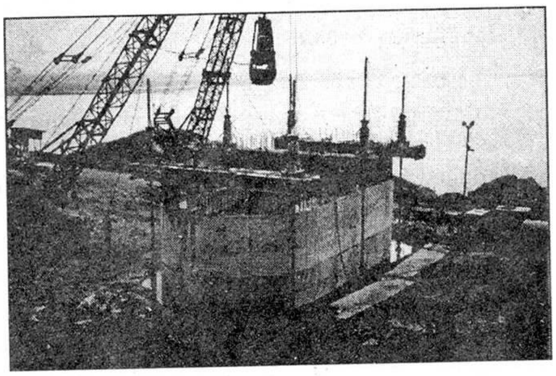


Photo 2 : A view of sinking of caisson in progress

Sub soil strata varies from fine sand at the top to clayey silt at the foundation level having silty sand and silty clay layers in between.

3. SPECIAL METHODS ADOPTED FOR SINKING

3.1. The following special methods for sinking of caissons were adopted individually or in combination as per site requirement depending upon the strata met during sinking.

3.1.1. Air jet method

Air jetting is a normal method used for reducing the soil friction on the outer periphery of caisson to facilitate the sinking. On this project, Air jetting system was adopted in a very different and effective manner so as to work all around the caisson periphery at the same time. It also provided scope to increase or decrease the area and extent of air jetting to suit the site requirements. In this method, PVC pipes were left on periphery of steining just touching the surface of formwork in first four lifts of steining after casting of curb. Rubber flaps were fixed on the pipes at 1 metre interval all along its length. After concreting and removal of formwork, 4 mm dia holes were made in PVC pipes and rubber flaps were fixed which were normally visible at the concrete surface. These holes act as nozzles during air- jetting whereas the rubber flaps protected the choking of these pipes. The pipes of each row were carried right up to the top in subsequent lifts of the steining.

When compressed, air is blown from compressors through these pipes, air penetrates in between the soil mass and the concrete surface and reduces the frictional resistance on the surface all along the periphery. Intensity and extent of air jetting can be controlled by operating one or more rows of pipes at a time or even on one side of the caisson. Normally, four horizontal rows in the lower portion of caisson are sufficient to loosen the soil around and above the curb portion which offers maximum soil resistance during sinking.

Air jet method works very well in cohesive soils. Pressure of air at any stage of sinking is kept 50% more than the water pressure at the bottom of caisson. Maximum air pressure is normally kept within 7 kg/cm^2. One or more compressors of 200 to 300 Cfm capacity are required for air jetting preferably with an air tank of 5 to 10 cu.m. capacity depending upon the site condition. Air tank should be capable to withstand a pressure of at least 8 kg/cm^2. Air Jetting alone was quite effective in early stages of sinking but was generally used in combination with jack-down method at deeper depths. It has been observed that by air jetting, frictional force on the outside surface of steining is reduced up to 20% of the anticipated load required for sinking the caisson. These pipes were later used for cement grouting of soil mass around the caisson after bottom plug to restore the soil friction around the caisson in the bottom portion below scour level.

3.1.2. Water jet method

The water jet method is used for cutting hard soil from inside the caisson and to remove soil below and around the curb and cutting edge portion. During dredging operation, the soil below and around the cutting edge and curb does not get removed specially in case of cohesive soils and hard strata. This soil is cut by high pressure water jet at a pressure of 100 to 150 kg/cm^2. It makes cutting edge free and allows the caisson to sink. This method is very effective in clay or hard strata, either alone or in combination with air jetting and Jack-down method.

3.1.3. Jack-down method

The basic concept of Jack-down method is to push down the caisson into the ground by applying load from top of the steining through jacks which take reaction from soil anchors. Soil anchors are first made at the predetermined locations in the bed as per requirement along periphery of the wall and load is applied by jacks through fabricated steel girders which are placed on top of steining. The load to be applied by jacks depends upon the size, shape and depth of caisson and the subsoil strata. The jacks are operated individually or collectively and load on each jack can also be varied to control the tilts of caisson during sinking. The soil anchors are basically friction piles and are designed to take reaction of jacks. However, friction is considered only in the length of anchors below final founding level of caisson. Soil friction around caisson is reduced by air jetting and soil below cutting edge and curb is excavated by water jetting as and when required. System is very clean, fast and effective and sinks the caisson in true vertical position with controlled tilts, shifts and rotation. Arrangement of Jack-down method as adopted at site is shown in **Fig. 3.**

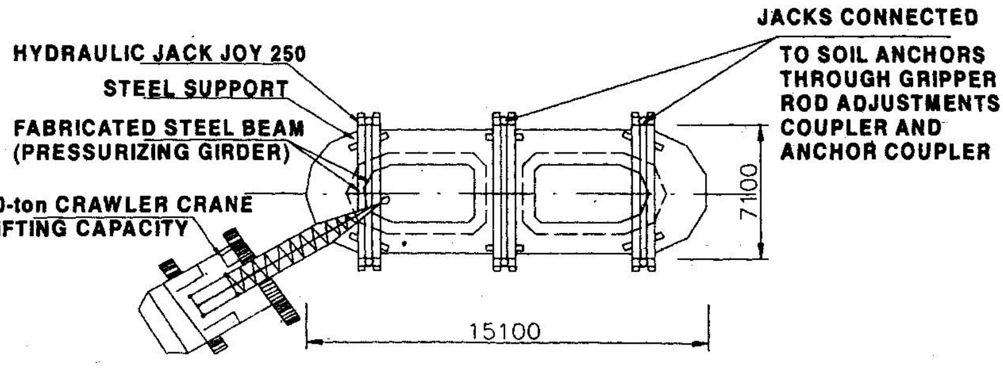


Fig. 3 : Arrangement of jack down method

The pressure on caissons were applied by six jacks of 250 Tonnes capacity up to a maximum load of 1200 Tonnes. The pressurization mechanism is shown in **Fig.4.**

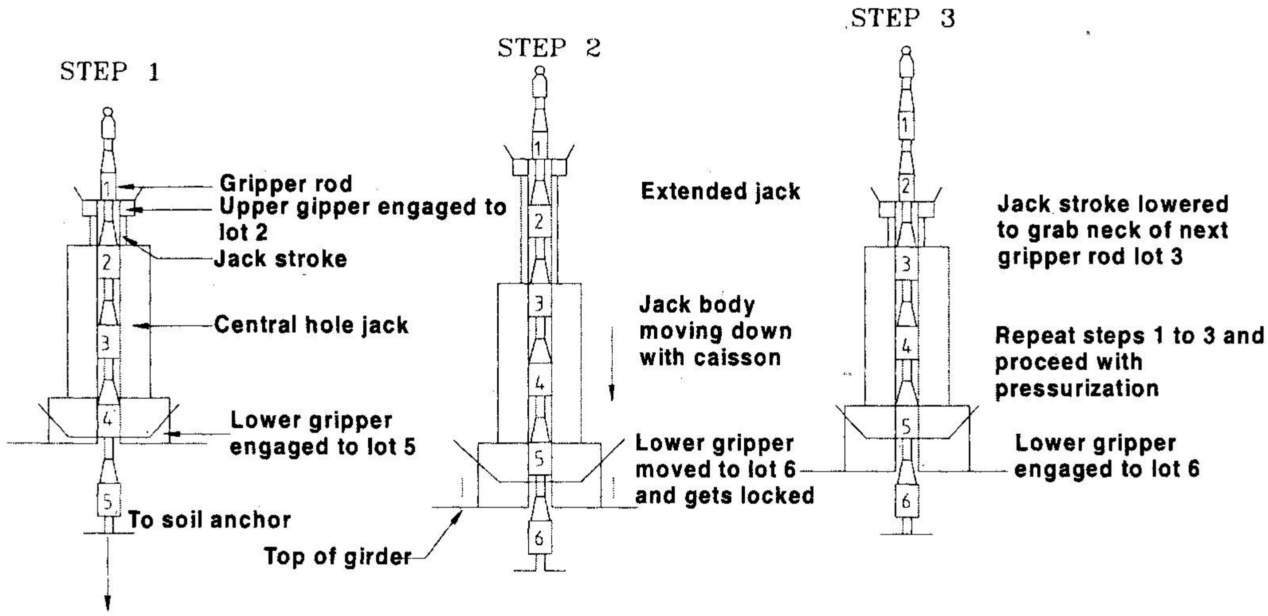


Fig. 4. Pressurizing mechanism to jack down the caisson

Jack down method consists mainly of the components as shown in **Fig. 5.**

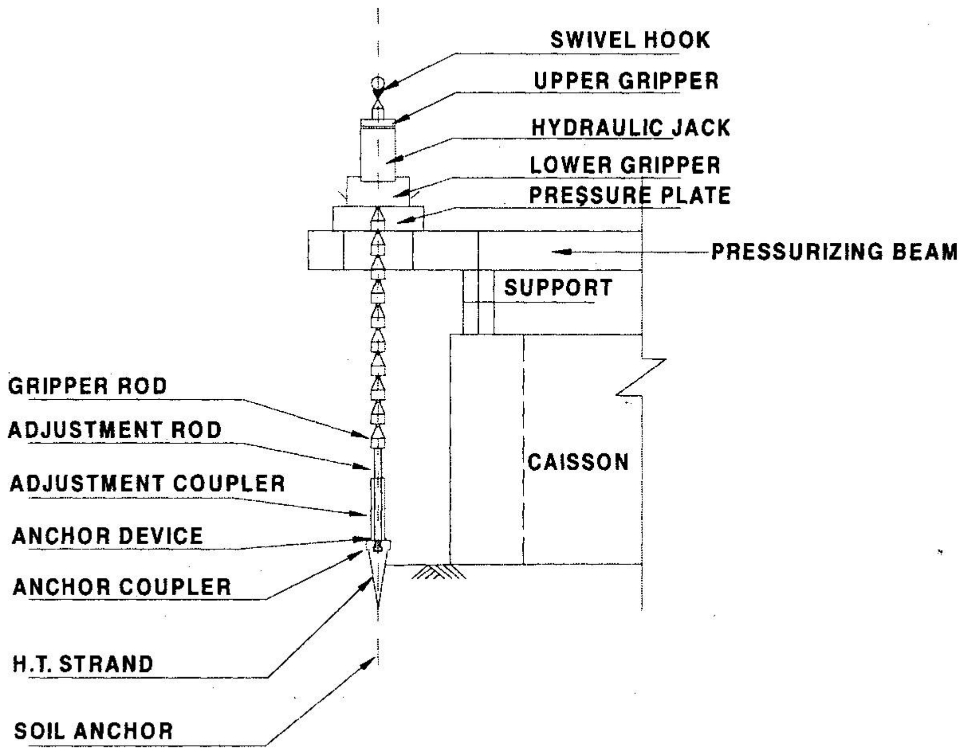
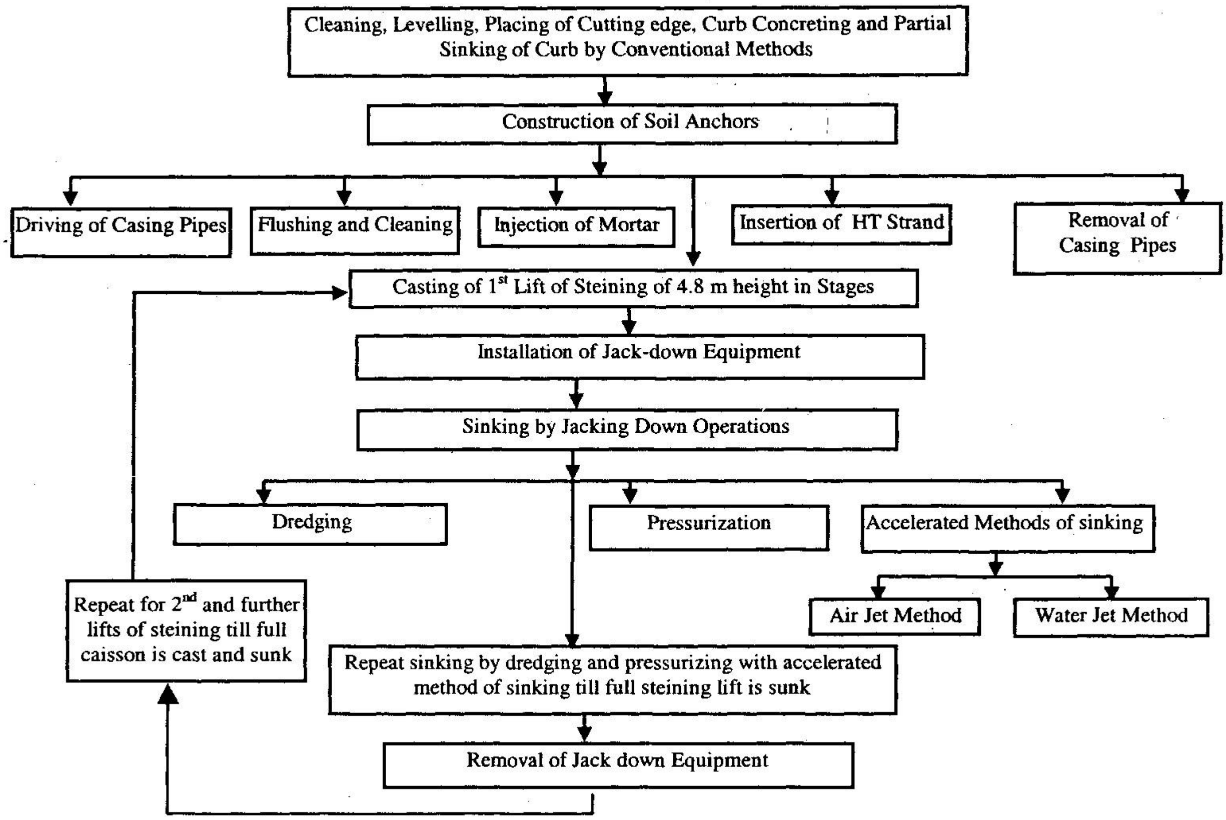


Fig.5. Components of jack down method

The sinking of 4.8 m lift of steining by Jack-down method for oval D-shaped caisson of 15.1 m x 7.10 m size has taken about 8 days, with an average rate of sinking of the order of 60 to 70 cm/day.

A typical flow chart of activities involved in construction and sinking of caisson by Jack-down method is illustrated in the following chart.

Flow chart for construction of caisson by Jack-down method



4. SAVING IN TIME AND COST BY ADOPTING JACK DOWN METHOD

Nizamudding bridge which was started in Feb. 1996 and was scheduled to be completed by March 1998, was completed in Feb. 1998 itself. Thus time and cost overruns which are generally experienced in most of the construction projects in our country were avoided. Early completion of the project was due to mechanization of site activities, specially the well sinking by jack down method. The sinking of 15.1 m x 7.1 m size oval shaped caisson within permissible limits of tilts and shifts by conventional methods of sinking is a very difficult and time consuming process. Therefore, it is the normal practice to adopt two circular wells of 6 to 8 m dia in place of single double D well of large size. The normal construction time of two single circular wells of 6 to 8 m ø dia upto well cap level with 36 m depth below LWL by normal method is about 240 days, while this large oval shaped caisson was completed in 130 to 135 days. Thus, the time required for completion of foundations of Nizamuddin Bridge by Jack down method was considerably less than that required by conventional method. For sinking of wells by conventional methods, the thickness of steining should be sufficient to sink the well under its own weight with little kentledge. By adopting jack down method, thickness of steining can be reduced as per actual design requirements due to its controlled operations. On Nizamuddin Bridge, RCC steining of 125 cm thickness has been provided while as per codal requirements, minimum thickness works out to 175 cm. Therefore, there was also substantial saving in concrete in well steining by adopting Jack down method.

5. CONCLUSIONS

The special method of sinking adopted on Nizamuddin bridge is one of the important factors for fast construction of the project. Fast construction and sinking of oval D-shaped 15.1 m x 7.10 m size caissons with permissible tilts and shifts could be possible only by Jack-down method of sinking supported by air and water jetting. The following conclusions can be drawn from the execution of this project:

- Jack-down method is simple mechanization of sinking process and can be adopted on projects specially with higher depths of foundations and large size of caisson in all types of strata.

- With mechanized system and controlled operations of sinking, speed of construction and tilts and shifts of caisson are effectively controlled which are the two major factors in construction of caisson foundations.
- Provisions in Indian codes for steining thickness are mainly based on the assumption that thickness of steining should be sufficient for self sinking of caisson to reduce sinking efforts and to avoid excessive loading during sinking. Therefore, considerable saving in concrete and thereby in cost can be made by reducing thickness of steining with Jack-down method of sinking.
- The faster completion of the project due to fast progress of sinking and construction of foundation shall considerably affect the benefit-cost ratio.
- Economics of Jack-down method is of course a point for study but it is only the initial investment in procurement of Jack-down equipment while the operational cost is very nominal. Though the soil anchors are non-recoverable, even then overall impact on the total cost of the project by adopting these mechanized methods after economizing the designs and with faster speed of construction shall be very nominal.
- Jack-down method is more effective in controlling the tilts and shifts in sinking of caissons and could be tried on important projects.
- Air jet method as adopted on this project is also a systematic and effective approach for reducing the soil friction around the caisson. This method can be used on all sites without any difficulty and with a little cost and efforts as it does not involve any special measures.

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UNIQUE FEATURES OF FOUNDATION NOS.17 & 18 - JOGIGHOPA BRIDGE

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S.A.Reddi, born 1933, is a Fellow of Indian National Academy of Engineers and of Institution of Engineers (I). During the last 40 plus years has been actively involved in the design and construction of more than 100 prestressed concrete bridges and is actively involved with BIS and IRC. Convenor of Foundation Committee of IRC involved in the revision of foundation codes

SUMMARY

The river Brahmaputra is one of the mightiest water courses in India. Bridging the river has always been a great challenge to Engineers. Apart from variable river course with frequent changes, the construction working season is very short, about six months in a year. The 2.30 Km. long rail-cum-road bridge is situated at Jogighopa, 150 Km. downstream of Guwahati. The bridge has 17 spans of 125m + 1 span of 94.6m and 2 shore spans of 32.6m each. The superstructure consists of 18.5m deep open web steel girders with the roadway on the upper deck and railway line through the lower deck. Double `D' caissons have been adopted for all except two foundations on uneven sloping rock and as such, special consideration was given in the design and construction of foundation Nos. 17 and 18.

Detailed soil investigations indicated that barring foundations 17 and 18, the caissons could be founded on compact sand at a depth of about 75m below the HFL. At locations 17 & 18 investigations revealed presence of rock strata at various depths. As per the original configuration of 125m spans, foundation Nos. 16 to 19 would have been affected by the rock profile. The rock level was close to maximum scour level and as such, caissons with no grip during maximum scour were unsafe.

2 x 260m cable-stay spans were earlier considered in order to reduce the number of foundations in rock. On detailed techno economic analysis, this was not found attractive. It was decided to provide 18m diameter RC caissons with central vertical diaphragm. The spans were readjusted to ensure that only two foundations are thus affected. The paper describes the special features of these foundations which includes solutions adopted for the first time anywhere in the world.

CHOICE OF SOLUTION

At locations 17 and 18, the rock at founding level was steeply inclined. The following options were considered : (a) piles enclosed in sheet pile cofferdam, (b) piles with diaphragm wall, (c) caissons with rock anchors, (d) caissons with piles and (e) caissons protected by boulder apron. A high level technical advisory group, after deliberations advised Option(d). The 18m diameter caissons were anchored into the rock with 12 Nos. of 1500mm dia RC piles through the steining (Fig.1). At the base, the critical load combination consists of (a) Vertical loads : 21438 t ; (b) Base moment : 223183 tm., and (c) Horizontal base shear : 4865 t. About 76% of the base area is under tension. The maximum pile capacity under tension is about 1450 tonnes.

Prefabricated annular steel caisson fabricated on shore was floated to the location and grounded by concrete filling. The steining was progressively built in 2.5m lifts. A 60 cu.m. capacity floating batching plant in conjunction with concrete pump and placer boom was used for concreting. The sinking was done by open grabbing with a 75 t floating crane.

CONSTRUCTION OF ANCHOR PILES (Fig. 2)

Casings - For anchor piles, 1.65m dia. holes were provided in the steining during construction. (Fig. 3). Sets of 10mm thick casings were fabricated in the yard and assembled in 10m long modules. The first module was lowered in the pile opening using a crane and temporarily supported at the top. Successive modules were erected, joints welded and lowered progressively till entire length of 50m weighing about 25t. was assembled. Thereafter, it was driven close to the rock surface by use of swing head oscillator and air lift.

Drilling - Wirth Reverse Circulation Rig was used to drill through the rock below the base of the caisson (Fig.4). The drill string consisting of a 13m long bottom stiff assembly weighing 15t was lowered and thereafter 3m long drill pipes were lowered alongwith stabilizers at 9m intervals through the clamping and lowering/lifting arrangements in the rig. The average rate of drilling was about 50 cm/hr in weathered rock and 20 cm in hard rock. A positive head of 2m - 3m was maintained in the borehole during drilling to preclude any possibility of sand ingress from outside.

Rock is cut by rotary motion of the drill under the down-crowd varying from 50 - 120 bars. The rig is fitted with a low speed high torque hydraulic motor (Fig.5), and Tungsten carbide button cutters. The pulversied rock is airlifted through the 200mm drill pipe and a hydraulic power pack. Compressed air is fed to the bottom of the drill assembly through twin 40mm ID steel pipes fitted on opposite sides of the main drill pipe.

Reinforcement and Concreting - The 60m reinforcement cage was lowered in 7 modules (total weight 25t) and anchored into the well cap at top. The second cage segment was wrapped by 5mm thick liner as permanent formwork from rock level to the bottom of steining. A flapper arrangement was provided at the top of the liner, which opens out under the pressure of fresh concrete and close the annular gap between the reinforcement cage and the parent 1650mm diameter casing to prevent leakage of concrete through the gap. The temporary casing was then retracted with the help of hydraulic jacks and cranes. 125 cu.m. of 30 MPa grade concrete per pile with 40mm rounded aggregates and 180mm slump was pumped through 200mm dia. tremie.

JET GROUTING - SOILCRETE

The Process - Due to steep rock profile, the caisson could not be advanced to the rock level and seated evenly on rock. Attempts to excavate the caisson to full depth and to clean the rock surface failed because of the inflow of sand through the gap between the cutting edge and the rock. This was closed by installing soilcrete columns by jet grouting. The soil is eroded by a high energy jet. The eroded soil and the injected cement water grout is mixed insitu to form solid mass columns, 1.3m dia., spaced at one metre centre, into a secant profile. There are 121 columns consisting of two rings, installed from rock level to approx. 2m above the cutting edge.

Bore holes for jet grouting were drilled from working platforms on steining top. A high pressure pump was used to flush during drilling with a cement water grout (W/C 1.0). On reaching the rock level, the drill bit is shifted to jetting mode and the soilcrete process is started. The drill rods rotate at a constant speed which is determined by nature of the soil. The rods are gradually withdrawn at a steady rate. The rotating grout jet erodes a cylindrical soil mass which is mixed with the grout itself. Surplus material rises along the drill rods to the surface and is discharged into the river.

Quality Control - Prior to the actual jet grouting operations trials on the river bank were undertaken by constructing soilcrete columns in shallow depths to freeze operational and quality control parameters. The trials indicated that column diameters of 1.2 - 1.5 m could be achieved and the following operational parameters were fixed (Table 1) :

TABLE 1

Parameter	Drilling	Jetting
Benotonite pumping pressure	8 - 20 bars	-
Air pressure	2.5 - 4.5 bars	6 hours
Grout pressure	-	360 - 380 bars
Drill rod rotation	50 rpm	12 rpm
Retraction	-	25 cm/min.

The following data was observed and recorded during actual construction :

- grout and air pressure during jetting
- grout consumption during drilling and jetting
- rotation and withdrawal speeds of drilling rods during jetting
- final penetration depth and top elevation of column
- specific gravity of grout/mortar when returned from borehole. This value gives an indication of the column diameter.

Due to the large depth, drilling accuracy was a crucial parameter. Drilling rig was carefully set up over the location determined by accurate survey, taking into account the actual tilt of the caisson. Drilling was done in a controlled manner. On reaching the final depth, the exact location was surveyed using a special inclinometer. At few locations, where deviation was excessive, the boreholes were abandoned and redrilling was done. After the survey, the jetting was taken up. The drilling and survey operations required about one hour, before starting jet grouting. This was too long a period to keep drill rods in cement slurry and as such bentonite slurry was preferred.

BOTTOM PLUGGING

(a) Sink the caisson upto 1m above rock.
(b) Stabilize the soil around well kerb by soilcrete columns.
(c) Remove the sand in the dredge hole by grabbing and air lifting and clean the entire area below the well kerb.
(d) Construct the bottom plug, and lay a RC slab on top of the plug after complete dewatering of the dredge hole.

EXTERNAL ANCHOR PILES

In the original design, it was assumed that the bottom plug of the caisson will have 100% contact with the rock. Any local cavity was to be grouted from the top. During the bottom plugging at location 17, it was observed that either due to high flood level or some crevices in the rock, fine silt was deposited at the rock - plug interface just before or during the concreting. It was decided to construct 8 Nos. of 1.5 m diameter external piles integrated with the well cap at top. These external piles were taken up as parallel activity during erection of deck and anchored into hard rock for 5m.

The well cap was constructed in two stages. During the first stage part of the cap having the same diameter of the well was constructed with the reinforcement projecting out. The extended cantilever well cap/pile cap was constructed in the second stage keeping 1.8m dia, hole for the pile.

In the soil above the rock level, boring was done by using oscillatory hydraulic piling rig (Casagrande) with custom built 7.5m long casings. As the casings were progressively lowered, additional casings were installed and the joints welded. Thereafter the Wirth piling rig with drill bit, 1300mm diameter was used for boring 5m in rock. Reinforcement cage of total length of 48 m was lowered progressively in five segments, welded insitu. The concreting was done by using automated batching plant, placer boom and concrete pump placed on a floating pontoon. The piles were integrated with the top of pile/well cap by a suitable anchor system. To improve the stiffness, a tie beam was provided at low water level.

CONCLUSION AND CREDITS

Completion of wells 17 and 18 required adaptation of innovative techniques for overcoming complex problems encountered under very difficult working conditions. The use of jet grouting technique for tackling foundation problems was successfully attempted at such great depth in the river bed probably for the first time in the world. The bridge owned by the Indian Railways and constructed by Gammon India Limited, has been commissioned in May 1998.

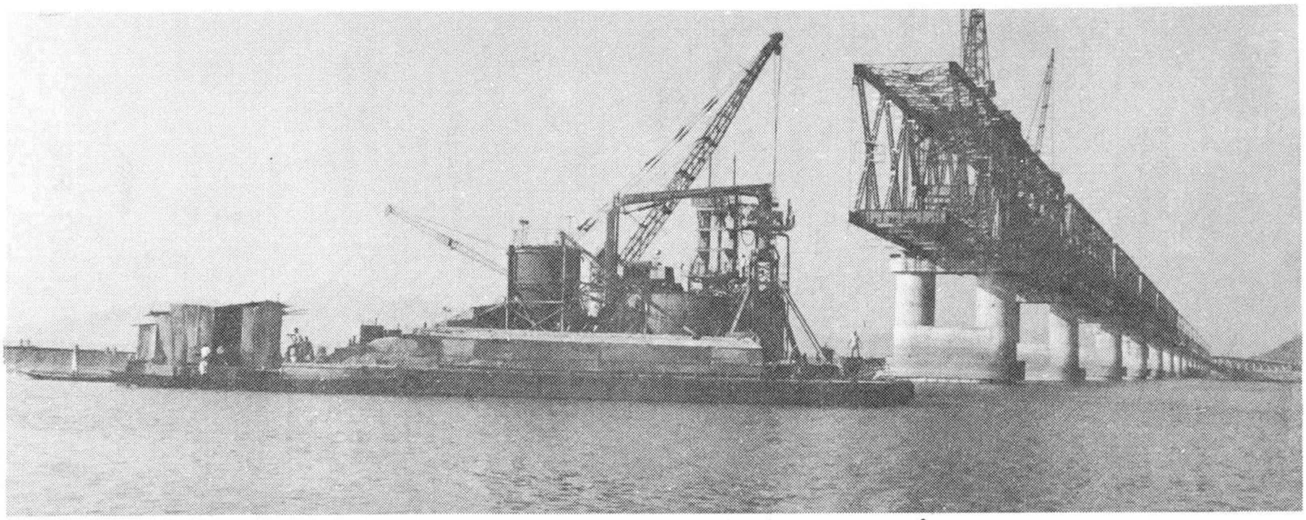

Fig.1 : Wirth Piling Rig in operation

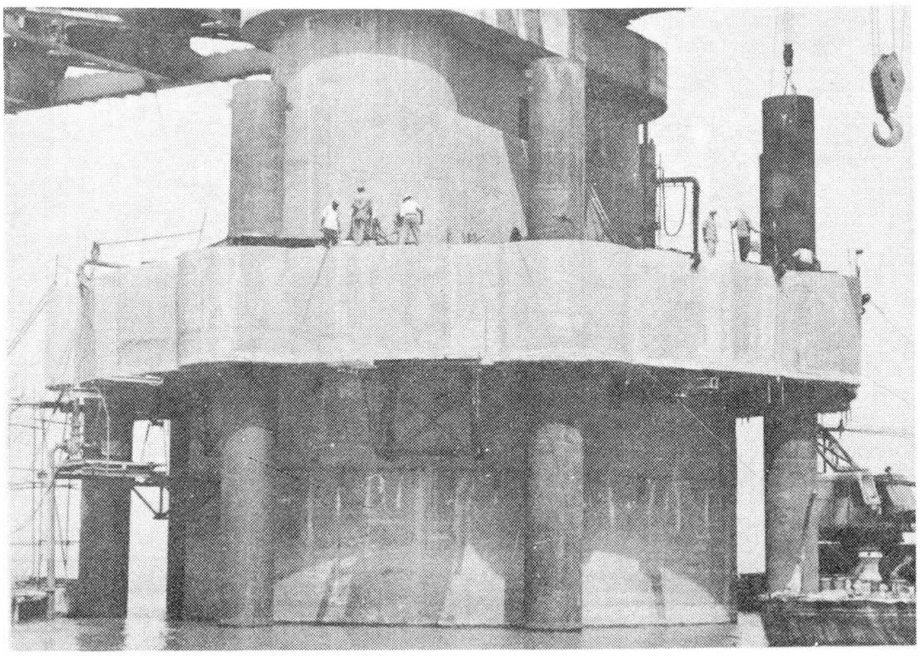

Fig.2 : External Piles

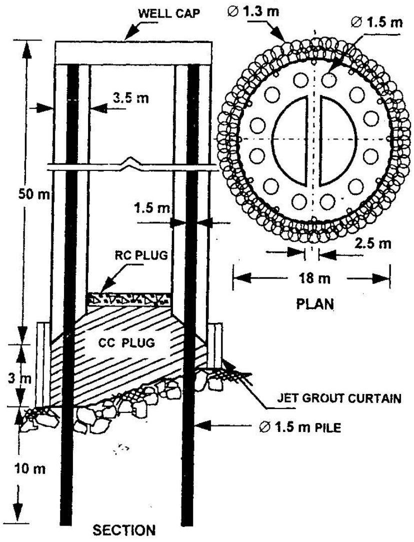

FIG. 3 JOGIGHOPA BRIDGE FOUNDATIONS 17, 18

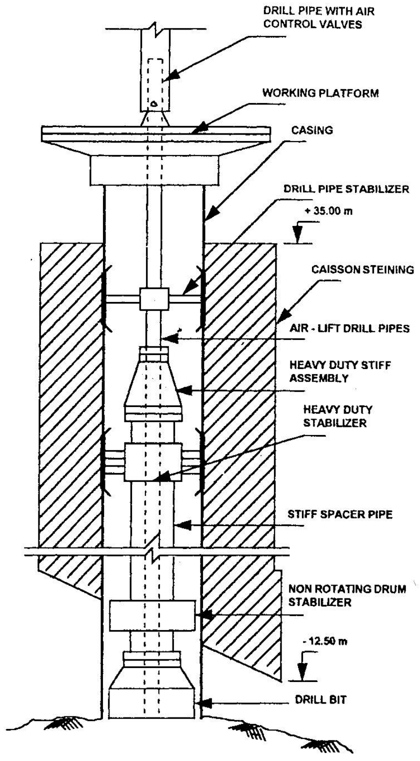

FIG. 4 WIRTH DRILLING RIG

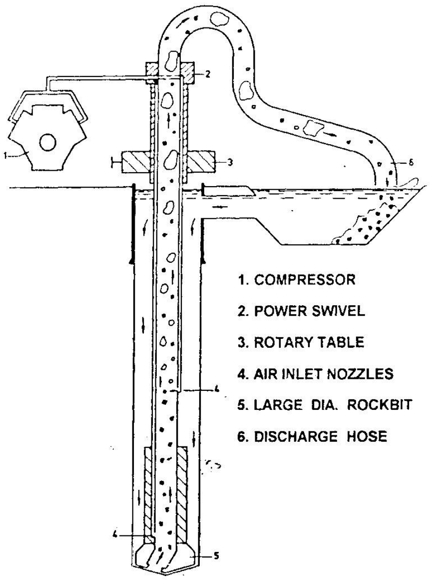

FIG. 5 AIRLIFTING MUCK

Well Foundation Construction in Bouldery Bed Strata – A Case Study

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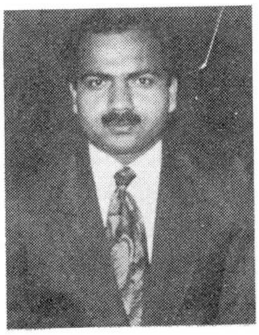


Graduated in Civil Engineering from Punjab Engineering College Chandigarh in 1986 and engaged in Bridge Construction with special experience of well foundation construction in boulderly bed using pneumatic sinking

SUMMARY

Bridge construction in boulderly bed poses challenge for engineers firstly due to difficulties in finalization of design depth of foundation due to lack of reliable formulas in present literature, secondly during construction stage due to difficulties in sinking of well foundation even with the use of pneumatic technique as the soil strata underneath is of heterogeneous character. Construction problems faced specially during sinking of well foundation due to mismatch of soil strata anticipated by SS1 and actually encountered at site. Open grabbing, diving and pneumatic sinking is required at various location to sink the well to desired level. A case study of well foundation construction in boulderly bed has been discussed in this paper.

Introduction

1. Bridge Construction in bouldery bed strata poses a challenge for the engineers for two basic reasons :

   (a) Due to non-availability of a proper formulae to calculate the scour depth.
   (b) Due to difficulties in construction while executing the work.

2. Most of the long span bridges are on well foundation specially, where there is a transition of the river from the hill area to the plains. Sinking of a well in such strata impedes the progress of execution due to the heterogeneous character of the soil and also due to the presence of large size boulders and wooden logs in the soil strata. A case study of well foundation construction in bouldery bed strata has been discussed in this paper.

Well Foundation – An Overview

3. The speed of construction of a well foundation depends upon the shape of the well the soil strata available and the equipment/plant deployed for construction of a particular type of foundation. The depth of a well foundation is finalised based on the scour and structural criteria and the higher of either of the two is adopted.

Essence of Design Parameters

4. The design parameters to be considered for a well foundation are hydraulics and soil parameters. The stratification of the soil below the proposed well foundation plays a major role while finalising the design and expediting the construction of the foundation. The hydraulic parameters considered are the silt factor, the bulk density, the angle of internal friction, the angle of wall friction and the cohesiveness of the soil. These are assessed based on the geological conditions of the strata. On assessing this data, the foundation levels are finalised. The correct assessment of the scour in soil is also very important. This is finalised based on available formulae and model studies as deemed fit and a judicious judgement of the Engineer, based on his past experience or similar structures.

Scour Depth in Bouldery Bed Strata

5. A sample cross section of a river, as available at various locations near proposed bridge site in bouldery bed strata is indicated at Fig (1). Due to the heterogeneous character of the bed material, it is difficult to apply any formula to obtain the actual likely scour. The IRC code has also clarified, that in such situations, a judicious choice of the Engineer based on his experience on similar structures at such locations may be applied. Based on the ground conditions and the application of the formula as contained at clause 703.2.2.1 – IRC-78, the following results are generally obtained for a particular bridge scour.

Depth of Water From HFL to LBL	Scour depth from HFL	Inference
H1	DSM1	(i) DSM 1 < H1 } Not a Practical (ii) DSM 1 = H1 } case (iii) DSM 1 > H1 } OK
DSM 1 – Mean Scour Depth		

6. It is further clarified that the results of DSM 1 and a silt factor have been indicated at Fig (2). Indicates that, after a silt factor `8', there is hardly any affect on DSM 1. The scour depth remains almost uniform even after an increase of the silt factor.

Method of Well Sinking

7. Well sinking in bouldery bed strata requires detailed planning which will vary from site to site. The soil strata obtained by the bore log must be examined. Basically, it is a guiding factor for the actual construction. A requirement for divers and pneumatic sinking can also be considered and necessary arrangements must be made to avoid any delay in the overall progress of the work.

Conventional Sinking

8. Conventional sinking is used for open wells. This is also known as grabbing. The material is taken out with the help of a grab. Progress in execution is expedited with kentledge if required and also with divers using jetting. Divers normally inspect the cutting edge and guide the crane operator Chiseling is also suggested if required. Explosives are also used in this technique, which results in shaking of the well and enhancing the rates of sinking. If the progress of sinking is very slow by this method, pneumatic sinking is restored to, to suit the site conditions and other related parameters.

Pneumatic Sinking

9. Pneumatic Sinking is resorted to in a situation, where dredging from an open well would cause loss of ground around the caisson or the soil underneath is full of boulders or the presence of clayey soil which poses problems for sinking the well. This technique is used for all types of soil, in case of bridges, where the progress of sinking by the conventional method is very very slow. This method is costlier but gives an output at a faster rate as compared to conventional sinking. Normally, on seeing the bore log details, the requirement of this technique is examined in advance and accordingly the design of the diaphragm or corbel slab and other necessary equipment for this technique are kept in readiness.

10. Penumatic sinking has the following advantages over the conventional method :

(a) The work is done in dry conditions and therefore control over the work on the foundation is better.

(b) Plumbness of the caission is easy to control as compared to the open caission method.

(c) Obstructions like boulders and logs can be removed. Excavation by blasting may be done, if necessary.

(d) Bottom plugging can be effectively done in pneumatic conditions.

A Case Study

11. A case study of well foundation execution of a major permanent bridge over a perennial river on bouldery bed strata is now discussed. The foundation depth of this bridge was more and overall perspective of well sinking in this case was a difficult one. The subsoil consisted of large boulders of 2 to 3 mtrs in diameter in the upper strata, to a depth of about 25 mtrs and below that, the soil was of silt sand aggregate matrix. In the initial stage, open grabbing with a crane-grab was resorted to for a depth of about 12 mtrs and subsequently, the rate of sinking was 10 to 20 mm per day. To expedite the progress pneumatic sinking was adopted, which led to increase in the progress to 150-900 mm. Pneumatic sinking can only be used up to a pressure of 3.50 kg/cm2. It is difficult for a human being to work beyond this pressure as per codal provisions.

Important Features of the Bridge

12. The important factors of the bridge were as follows :

(a) Length of the bridge - 704 Mtrs

(b) Type of Bridge - Prestressed concrete single cell box girder of balanced cantilever construction.

(c) <u>**Foundation :**</u>

(i)	Type	- Circular well
(ii)	Outer diameter	- 11.70 Mtrs
(iii)	Inside diameter	- 6.64 Mtrs
(iv)	Steining thickness	- 2.53 Mtrs
(v)	Well crub height	- 4.5 Mtrs
(vi)	Angle of cutting edge	- 33 degrees
(Vii)	Grade for Steining Concrete	- M25

Set up of Equipment and Plant

13. The layout ensured minimum movement of material, equipment and personnel as well as proper drainage of the area. Wind conditions were taken into consideration in operation of some equipment, for example, the operation of the tower crane which was not possible in heavy winds. Supporting facilities such as generators, office stores etc. were located away from the path of the dusflow. Adequate space was provided for handling and storage of raw material as well as for finished products. Wherever practicable, a separte service road was provided for incoming material and outgoing products.

Bore Log Details

14. Before starting the work at this location, bore log details of the location were examined and accordingly, the strategy for sinking the well foundaiton was planned. From the bore log details, it was obvious that sinking a well in the prevalent soil strata with open grabbing may not be possible due to packed bouldery strata as shown in the

Figs (3) & (4). Keeping in view, the likely strata, the bed well was designed for pneumatic pressure also.

Conventional Sinking

15. The construction of the well foundation at location was started in Jan 1980 with the placing of the cutting edge at a ground RL of 160.735. Conventional sinking was adopted with the use of Tata Crane having a grab of 1.2 cum capacity and divers were also deployed, after reaching considerable depth, to expedite the rate of sinking. Whenever a sizeable sump was made and the cutting was cleared by divers, blasting was resorted to, which shook the well and finally led to sinking the well (Fig. 5). This work was continued till June 90 and an average rate of sinking observed during the period was approximately 7 cms per day. But the rate of sinking during the last month before suspending conventional sinking was 1 cm. It was felt that the further rate of sinking would be extremely slow and it was decided to start with pneumatic sinking.

Pneumatic Sinking

16. Pneumatic sinking was attempted wef 4^{th} Jul 90 at am RL of 145.360 mtrs. The steel diaphragm was placed at a gauge height of 14 mtrs. This technique was continued upto a design foundation level of 125 mtrs (Fig 6). During the course of sinking, boulders upto 2 mtrs size were encountered, which slowed down the rate of sinking, as boulders more than 50 cms size could not be taken out because of the limitation of the muck bucket. There was also a variation in the water level of the river from 153.500 mtrs to 163.00 mtrs during the year. It was obvious that the deeper the digging, grater was the pressure and progress reduced considerably which further led to increase in compression and decompression time. However, limited progress was assured in the technique. Out of a total sinking of 35.895 mtrs, 15.350 mtrs was executed through conventional means and the remaining 20.300 mtrs to 163.00 mtrs during the year. It was obvious that the deeper the digging, grater was the pressure and progress reduced considerably which further led to increase in compression and decompression time. However, limited progress was assured in this technique. Out of a tital sinking of 35.895 mtrs, 15.350 mtrs was executed through conventional means and the remaining 20.300 mtrs was by pneumatic sinking. The output of sinking depends upon various factors which include the size of the bucket, the depth of the well, the type of strata, the water head, the size of the shaft and the airlock. The team of personnel inside the working champer comprised of one Engineer, two supervisors and twenty labourers. One mtr of sinking resulted in required 110 m3 of material to be evacuated, which in turn required 440 buckets for dredging out the material, as the capacity of one bucket was 0.25 m3. The progress of work reduced with the depth of sinking. However, a percentage of acceptable progress was achieved within a specified time. The rate of sinking reduced with progressive sinking, as at an increased pressure, the progress of sinking reduced .

SL NO.	Total Sinking achieved (Mtrs)	Average rate of sinking per day
1.	Conventional Sinking : 15,350 mtrs	7 Cms
2.	Pneumatic Sinking : 20.455 mtrs	4 Cms
	Total Sinking : 35.805 mtrs	

Bottom Plug

17. The total sinking of 35.805 mtrs was completed and the position of the well was as under :

(a)	Total steining from ground level	-	35.805 mtrs
(b)	Total steining cast	-	27.000 mtrs
(c)	Well crub height	-	4.500 mtrs
(d)	Gauge height of well	-	31/430 mtrs

18. The last leg of sinking was completed in pneumatic sinking and it was planned to plug the well in the present condition only, as this would be helpful to remove the steel diaphragm placed inside over working chamber.

19. The concrete was transported to the airlock and subsequently to the working chamber, where it formed a part of the bottom plug. This process was repeated a number of times, as in one operation only one cubic mtr of concrete was possible. Out of total height of 5 mtrs., 2 mtrs was completed in pneumatic condition and the remaining was executed in open condition after removing the diaphragm.

Sand Filling

20. Sand was taken from the river and cleaned before transportation. This was finally lowered through a chute placed on the top of the steining and subsequently the top plug and the well cap were cast.

Construction Problems

21. The following problems were encountered during the construction of the bridge :

(a) The sinking of the well became very difficult due to the presence of large size of boulders in the strata. This led to a considerable slow down in the overall progress of the bridge.

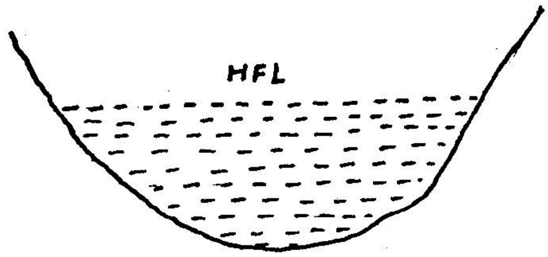
(b) Basically, there were difficulties in finally deciding the foundation level on such strata. This put the decision making body into a considerable dilemma and led to delay in progress.

(c) Due to heavy rainfall in the area, a considerably reduced working period was available in the region, which led to delay in overall completion of activities.

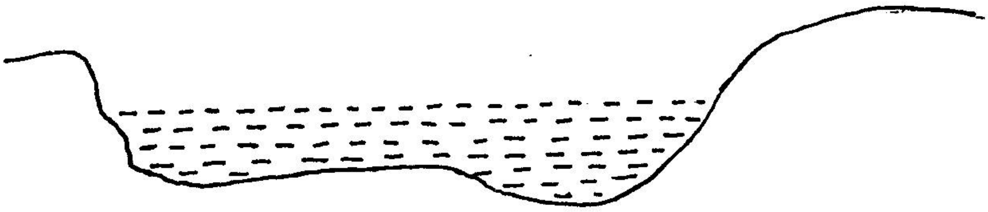
Conclusion

22. Construction of a well foundation in bouldery bed strata calls for a dedicated effort on the part of the executives. The data of the soil strata encountered at the site must be kept in detail for each meter and if required a review of the foundation must be carried out, based on actual soil parameters obtained during sinking.

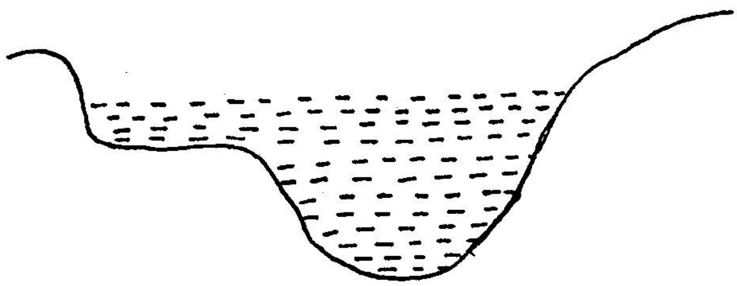
FIG 1



A. SEASONAL RIVER STEEP SLOPE NEAR HILLS



B. FLAT SLOPE



C. PERENNIAL RIVER WITH SIZABLE DISCHARGE

CROSS SECTION AVAILABLE IN BOULDERY BED

FIG 2

SILT FACTOR AND MEAN SCOUR DEPTH FOR A PARTICULAR DISCHARGE (100 m^3/sec/m)

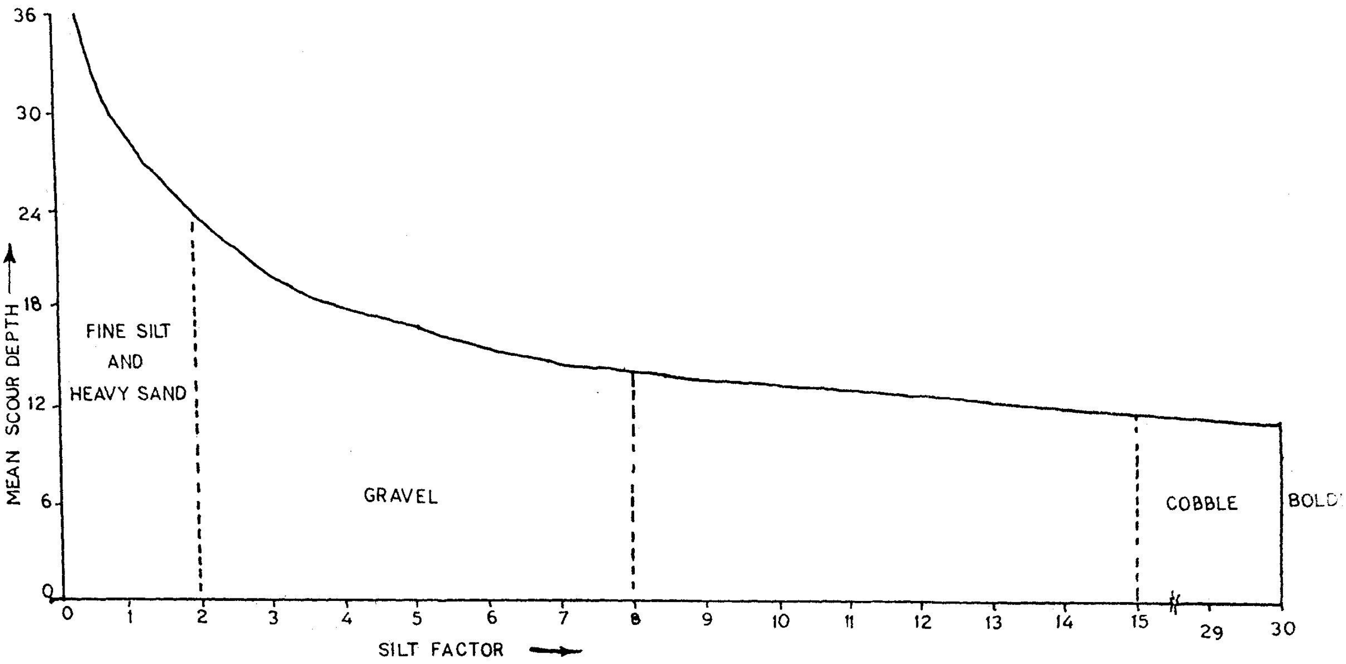


FIG 3

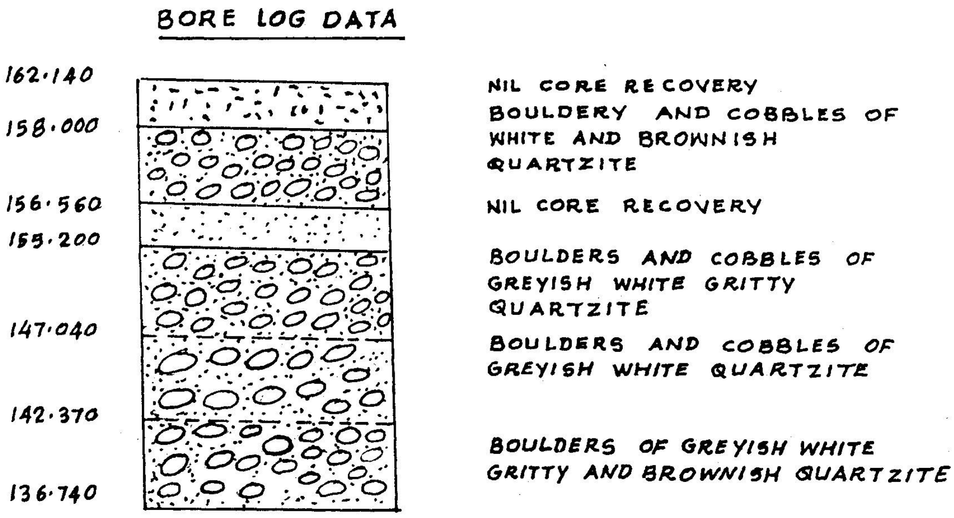

BORE LOG DATA
162.140
158.000
156.560
155.200
147.040
142.370
136.740
NIL CORE RECOVERY
BOULDERY AND COBBLES OF WHITE AND BROWNISH QUARTZITE
NIL CORE RECOVERY
BOULDERS AND COBBLES OF GREYISH WHITE GRITTY QUARTZITE
BOULDERS AND COBBLES OF GREYISH WHITE QUARTZITE
BOULDERS OF GREYISH WHITE GRITTY AND BROWNISH QUARTZITE

FIG 4

SOIL STRATA ENCOUNTERED

RL	PICTORIAL DESCRIPTION	STRATA DESCRIPTION
160·735		STRATA COMPRISING OF SAND SILT MIXED WITH 60% BOULDERS SIZE VARYING FROM 300 mm TO 600 mm
150·780		
145·360		SANDY SOIL MIXED WITH 60% BOULDERS SIZE VARYING FROM 300 mm TO 600 mm
141·180		SANDY SOIL MIXED WITH BOULDERS VARYING FROM 300 mm TO 1500 mm.
130·730		COMPACTED STRATA COMPRISING OF RED CLAYE SANDY SOIL : 40% BOULDERS UPTO 300 mm: 30% BOULDERS 300 mm TO 700 mm : 30%.

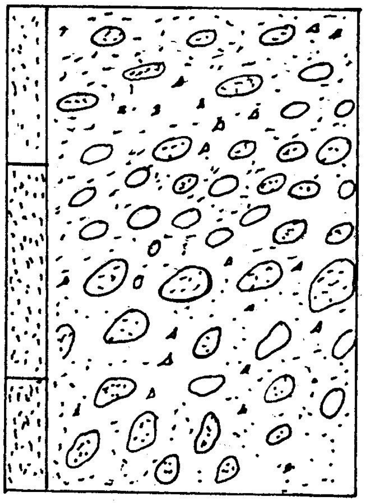
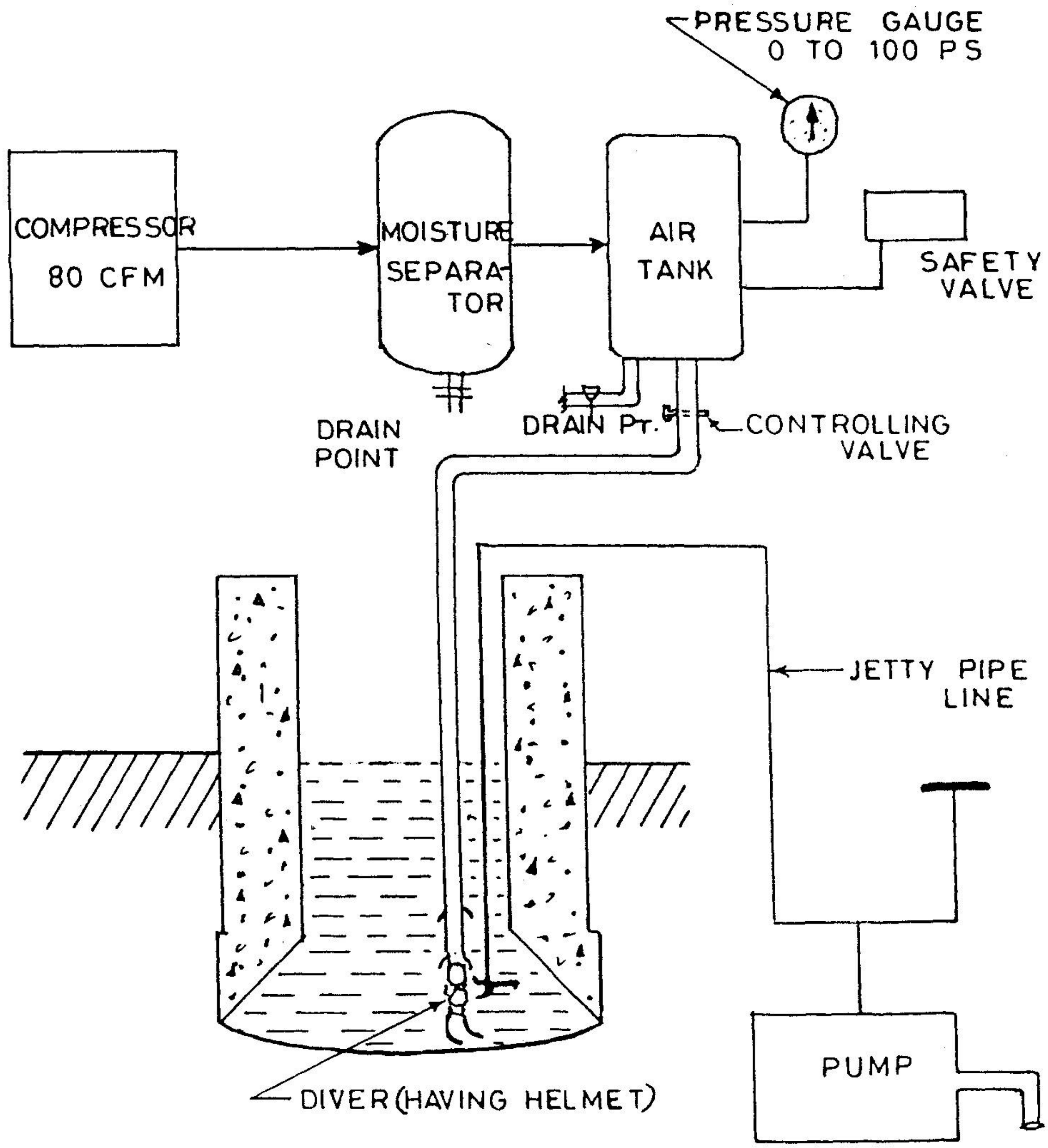
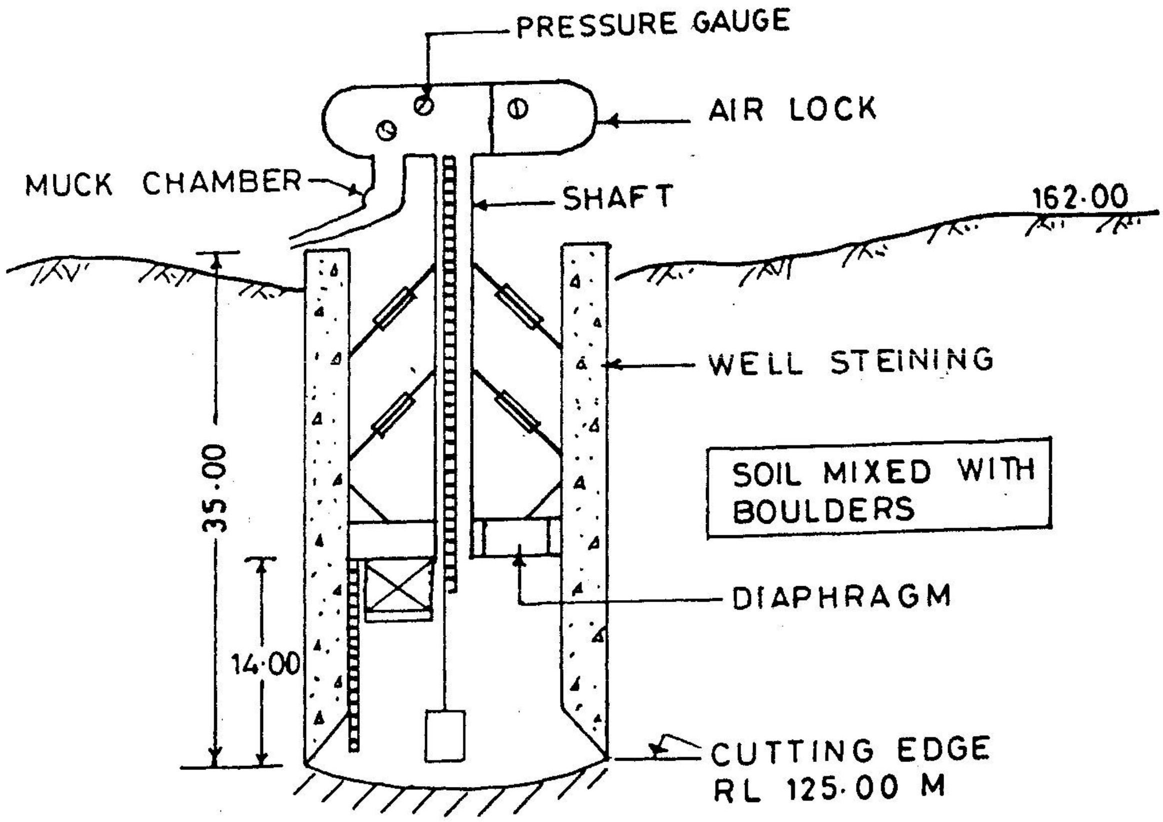


FIG 5



DIVING ACTIVITIES

FIG 6



FOUNDATION RL REACHED WITH
PNEUMATIC SINKING

CONSTRUCTION OF ANCHORED CAISSONS FOR A MOTORWAY VIADUCT

Nader HALIMEH
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Dar Al Handasah
Consultants
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SUMMARY

Caissons are a principal alternative to piles for bridge foundations, particularly where piles would have to be drilled or driven into ground containing large boulders or into rock of uncertain quality that might require further investigation during construction. A further advantage that caissons provide is more confidence, as the excavated surface can be closely inspected, and better quality concrete can be obtained, eliminating the need for integrity testing.

Caissons, some circular, others elliptical, provided the deep foundations for all piers and abutments of Viaduct V5, where the bed rock is overlain by several metres of slope debris. Step-by-step excavation was carried out by a hydraulic back-hoe excavator lowered down the caisson. Light wire mesh with shotcrete was used to support the sides of the circular caissons through slope debris, while reinforced-concrete ring beams supported by pre-stressed cable anchors were used in each excavation stage of the elliptical caissons. The anchors were installed using a drilling rig also lowered down the caisson. The design stressing force was generally achieved, except for the second stage of two of the caissons, despite the grouted bond length being increased. This problem was overcome by post-grouting the anchors, and accepting a lower force for the cable anchors of that stage. Proof tests on the pre-stresed cable anchors were performed at each stage of excavation. Inflows of water during the rainy season caused some problems, and interrupted construction of one of the deepest caissons. The caisson foundation was designed to be cast for the full cross-sectional area in vertical lifts, and the temperatures was monitored to assure quality. Heavy reinforcement was provided, to satisfy the minimum requirement of the AASHTO. The Paper gives some suggestions and recommendations based on experience gained.

1 INTRODUCTION

Viaduct V5 is one of the major viaducts presently under construction for a motorway in the Middle East. This section of the motorway is 258 km long with three lanes in each direction, and involves a number of large structures including fourteen viaducts.

Viaduct V5 is 500 m long, comprising nine spans, with a maximum span of 110 m in each carriageway, see Figure 1 below. The entire design and concept is governed by geological factors, as described below, which necessitated the use of caisson structures to form the deep foundations for all piers and abutments. Part of the viaduct super-structure is constructed of pre-cast beams, the remainder of structural steel box girders; accordingly, two different types of pier columns and foundations were required. Two types of caissons were designed, one circular, 6 m in diameter, and the other elliptical, 12 m by 8.5 m; the elliptical shape was chosen to reduce the obstructing effect that might occur in the event of a landslide. The caisson depth varies between 10 m and 31 m.

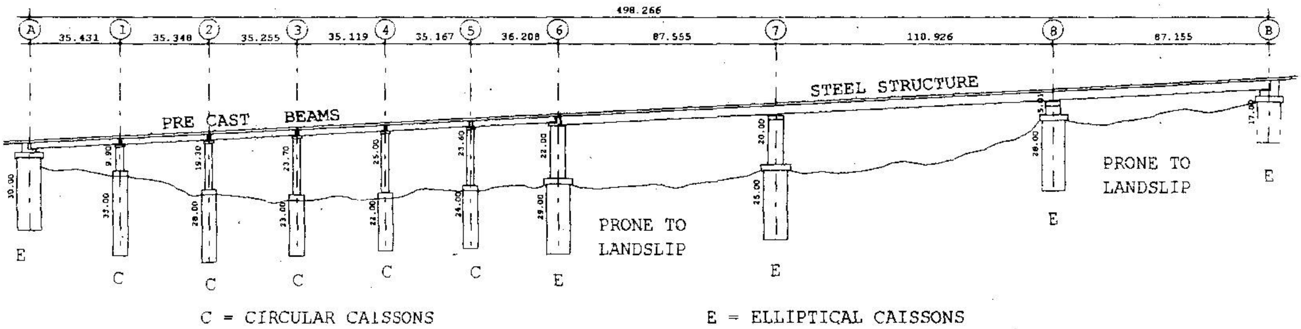


Figure 1: Longitudinal section of Viaduct V5.

2 GEOLOGICAL CONDITIONS

Situated in an area subject to earthquakes, the viaduct runs roughly from West to East sidelong across a steep slope which falls towards the viaduct from the South, and parts of the thick layer of unstable slope debris that makes up this slope have slid towards the viaduct in geologically very recent times; the viaduct spans over these landslide areas. The bed rock in the region is disturbed by faults, folding and dykes. The influence of fault- and dyke-related movements was minimised by shifting the alignment towards the North. However, the extension of the major faults which prompted the slide cross the viaduct alignment between Pier 6 and Pier 7 and between Pier 8 and Abutment B. These fault zones are associated with clayey fault gouges and completely decomposed dyke material. The slope debris comprises sand and gravel in an open matrix of clay . The caisson foundations went through this and down to sound strata consisting of alternate strata of thickly-bedded sandstone and thinly-bedded siltstone - sandstone. In general, this bed rock is slightly weathered and slightly to highly fractured.

3 EXCAVATION AND GROUND SUPPORT SYSTEM

3.1 Preparatory Excavation

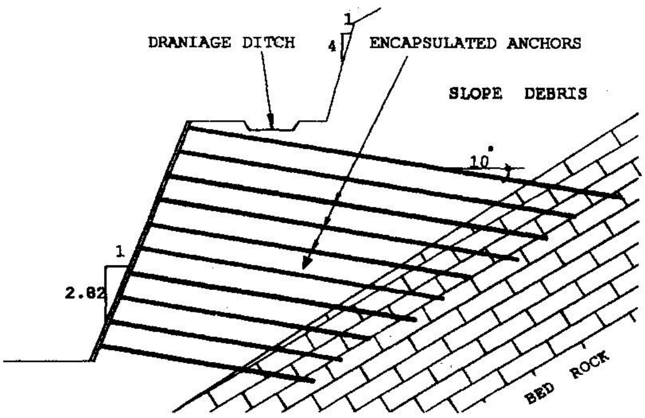


Figure 2: Slope formation.

Preparatory excavations were made in the slope debris in order to obtain a level working platform for construction of the caisson proper. These excavations were carried down step by step, each step being supported by shotcrete reinforced with wire mesh and retained by encapsulated (passive) anchors comprising 26.5 mm diameter bars of 835/1030 U.T.S. steel inserted into a 130 mm diameter hole. The anchors were embedded three metres into the bed rock, as shown in Figure 2. Two inclinometers were installed to monitor any movement during excavation.

3.2 Caisson Excavation

Excavation was carried out, using a hydraulic excavator, the use of explosives was rejected both from fear of precipitating further landslips, and to prevent loosening of anchors already installed for previous stages. The ground was removed down to the level of the next support stage; wire mesh was applied to the exposed ground that formed the wall of the excavation, and then shotcrete was applied as shown in Figure 3 on the next page. In the circular caissons, shotcrete 150 mm thick was sprayed onto the exposed ground, increased to 200 mm thick when more than 10 m below the top of the caisson. In the elliptical caissons, in addition to 200mm of shotcrete, a reinforced concrete ring beam was cast at the bottom of each excavation stage, through which pre-stressed cable anchors (active) were installed as described in Paragraph 3.3 below. These beams and anchors were only required through the slope debris; in rock, ring beams were formed in shotcrete. Drainage pipes , as shown in Figure 4, were installed through the shotcrete of alternate vertical panels, of each caisson, to prevent accumulation of ground water pressure behind the shotcrete.

The bigger the diameter of a caisson, the easier it is to excavate, but heavier support will be required. The deeper the caisson, the more difficult it is to construct - a heavy crane is required for lowering the excavator. A deeper caisson is also more likely to suffer more from water infiltration. A deep caisson of small diameter may also generate a ventilation problem, as well as suffering from congested working space.

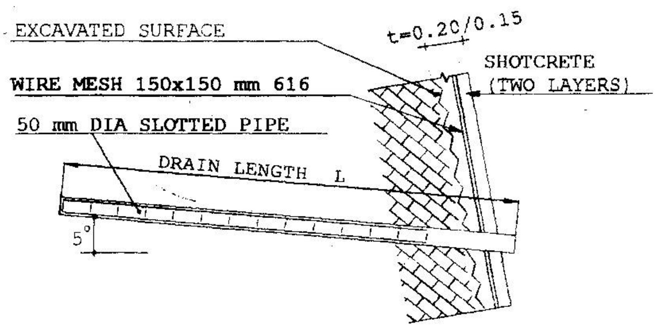
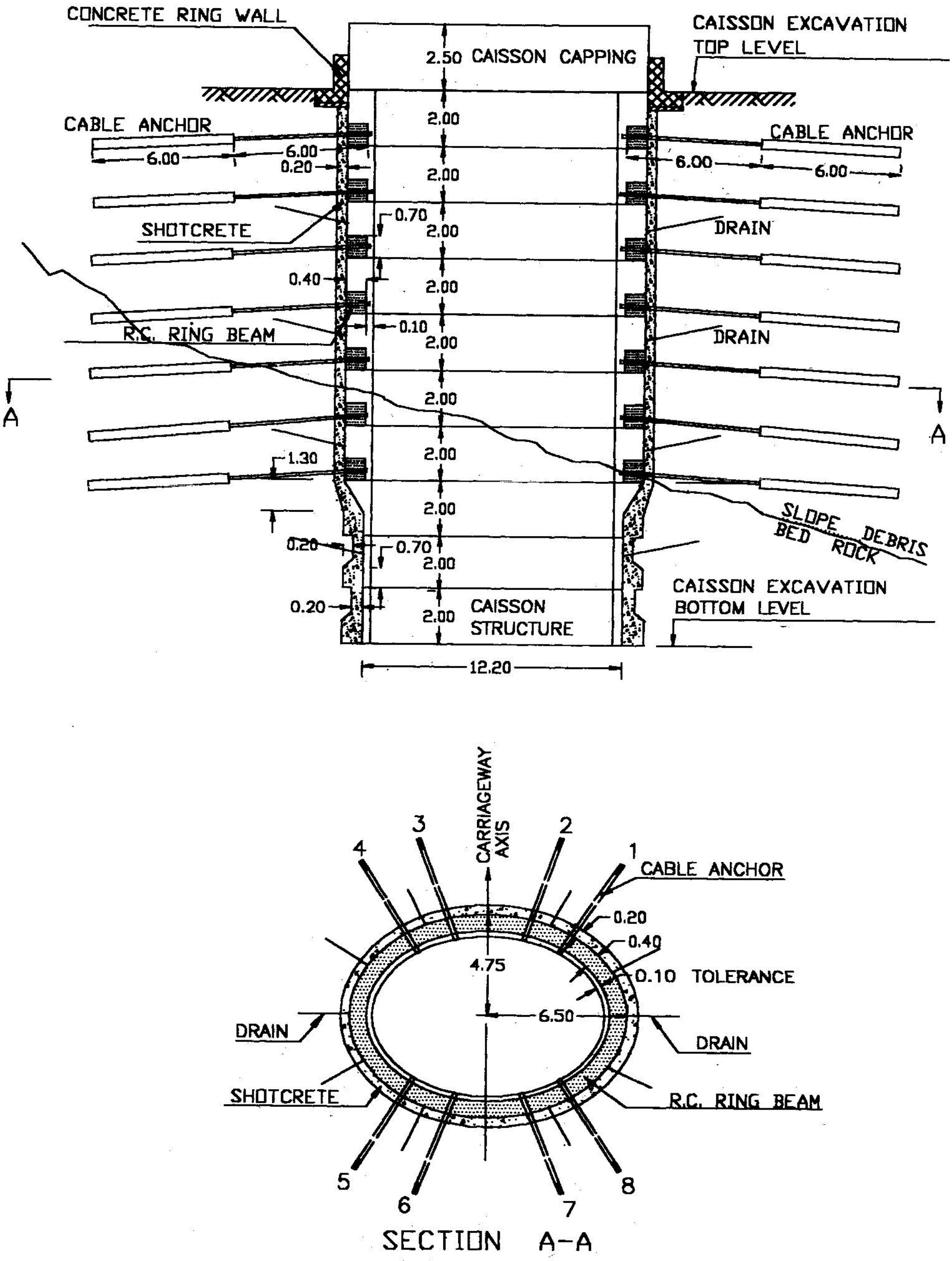


Figure 4: Drainage behind shotcrete lining.

The accumulation of ground water draining into the excavation was a cause of delay in construction of the deepest caissons; a system of four well-points outside and on the axes each caisson had been intended at the design stage, using 150 - 200 mm diameter pipes with submersible pumps, but this could not be adopted, because such pumps were not available. Although the Contractor used a sump at each level of the excavation, this proved to be



NOTE : ALL DIMENSIONS ARE IN METRES

Fig. 3. Plan and section of an elliptical caisson showing excavation stages and the peripheral supporting system.

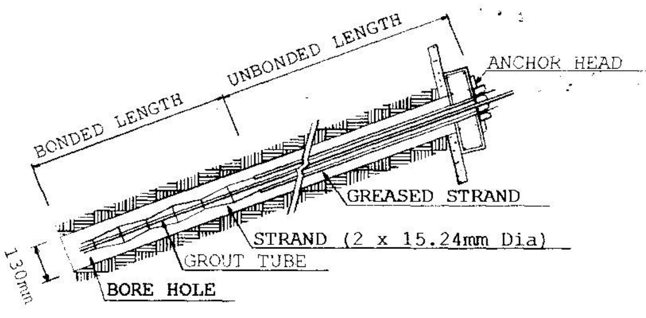


Figure 5: Pre-stressed cable anchor details.

insufficient to prevent work from being stopped during the rainy season.

3.3 Pre-stressed Cable Anchors

Eight cable anchors were used at each stage of excavation of the elliptical caissons, as shown in Figure 3, each anchor had two strands 15.24 mm in diameter as shown in Figure 5. The drilling of the 130 mm diameter holes was performed by a SoilMech SM 305 drilling rig, lowered down to the current floor of the excavation. The cable anchors were assembled on site, and inserted into the holes, after which grout was pumped in to form an anchorage, from a grouting plant located on the surface nearby. After the ring beam had gained sufficient strength, the anchors were stressed. The imposed stressing force was 150 kN for the top row, and 250 kN for the remainder; however, for some anchors at the second level this could not be achieved, even after the bond length was increased from six to ten metres, probably due to excessive ground water. This was overcome by accepting 150 kN for these anchors as well, and post-grouting them. One anchor at each level was tested in accordance with the relevant AASHTO specification.

4 CONCRETE

The entire cross-section of the caisson was filled with 22.5 N/mm² concrete, i.e. no internal shutter was used. The Contractor originally proposed to cast each caisson in one continuous operation, but this was rejected, and the concrete was cast in lifts of 2.05 metres in order to prevent a build-up of heat. The controlling factor was to ensure that the temperature differential between successive lifts did not exceed 20°C; however, this proved to be very slow, and as limestone aggregate was being used, the differential was increased to 40°C, as recommended by Neville & Brooks. Five rows of 32mm diameter bars were used as the main vertical reinforcement for the full height of the caisson, tied to 16 mm diameter spiral reinforcement.

5 DISCUSSION

"Because of the very great complexity of all ground problems, it is very dangerous to carry out a design prior to construction and never modify it" (Hanna, 1982). The designed depths of four of the caissons of Viaduct No. 5 were modified following close examination of the rock surface exposed as the excavation progressed. This was also the case with other viaducts in the Contract, and in some cases it was found necessary to carry out further site investigation, including the drilling of extra boreholes, for foundations where these were to rest on bored piles.

In general, tie-back walls are constructed in soft ground such as clay, sand, gravel and shales. The caissons for the piers adjacent to Abutment A of Viaduct No. 5 were six metre diameter circular caissons, as shown in Figure 1. When the Contractor came to create Abutment A itself, for each carriageway it have been logical to have two similar caissons, but he elected instead to have one larger 12 x 8.5m elliptical caisson with anchor-supported walls because the excavation of the smaller circular caissons at the adjacent piers had proved to be slow. Hanna points out that for any specific problem, and the above is a good example, a large number of possible solutions is often available, and selection is usually controlled by factors other than technical.

6 SUGGESTIONS AND RECOMMENDATIONS

1. The force obtainable in the anchors in the top four metres of an anchor-supported caisson will be less than for anchors further down.
2. For cable anchors in wet slope debris, it may be necessary to take extra measures to obtain the required loading, in which case a post-grouting system will give better results than increasing the bonded length.
3. Since the estimation of bond magnitude is uncertain, field anchor tests should always be carried out in order to confirm the bond values used in the design.
4. For caissons constructed by similar methods, eight metres is considered to be the smallest diameter that will avoid construction problems caused by limited working space and poor ventilation.
5. The shape of the elliptical caissons and the use of pre-stressed anchors for this viaduct were enforced by the existence of landslides; otherwise. the use of controlled blasting and soil nails would have been more economical.

7 ACKNOWLEDGEMENT

The Author would like to thank his colleague, Mr. Robin Clay, Chief Tunnel Supervisor, for his assistance during the preparation of this Paper.

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