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Leere Seite
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Page vide

Design of Foundations for the Storebælt East Bridge

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Mr. Pedersen has been involved in design of major bridges and offshore concrete structures for more than 20 years. He has specialised in design of large pylon and anchor block structures in high quality concrete for cable stayed and suspension bridges utilising advanced finite element design models. Mr. Pedersen has been in charge of the concept development, tender design and detailed design for the main bridge substructures of the Storebælt East Bridge.

SUMMARY

The fixed link across the 18 km wide Storebælt Strait in Denmark consists of three major projects; an 8 km bored railway tunnel and a 6.8 km high level motorway bridge across the eastern channel of the Strait and a 6.6 km low level bridge for combined railway and motorway traffic across the western channel. Navigational considerations have been very much influential in the design of the East Bridge as the main navigation route between the North Sea and the Baltic Sea passes the Eastern Channel of Storebælt. Other important design considerations have been environmental aspects like maintaining unrestricted flow of water to the Baltic Sea and not least the aim to establish an elegant bridge perfectly incorporated in the landscape. The paper describes the concept development and final design of the major structural components of the East Bridge substructures in view of these considerations. In particular, the design of the huge pre-fabricated foundation caissons placed on stone beds compacted under water is addressed. The structural calculations comprised advanced models for determination of effects from ship impacts as well as finite element models capable of handling built-in stresses in the structures during all construction phases.

1 INTRODUCTION

The Storebælt East Bridge comprises a major suspension bridge with a free span of 1624 m, the second longest in the world, and approach bridges with spans of 193 m. The total length of the bridge is 6790 m and the navigational clearance in the main span is 65 m, see Fig. 1.

The East Bridge substructures consists of three major structural components

- the pylons,
- the anchor blocks and
- the approach bridge piers.

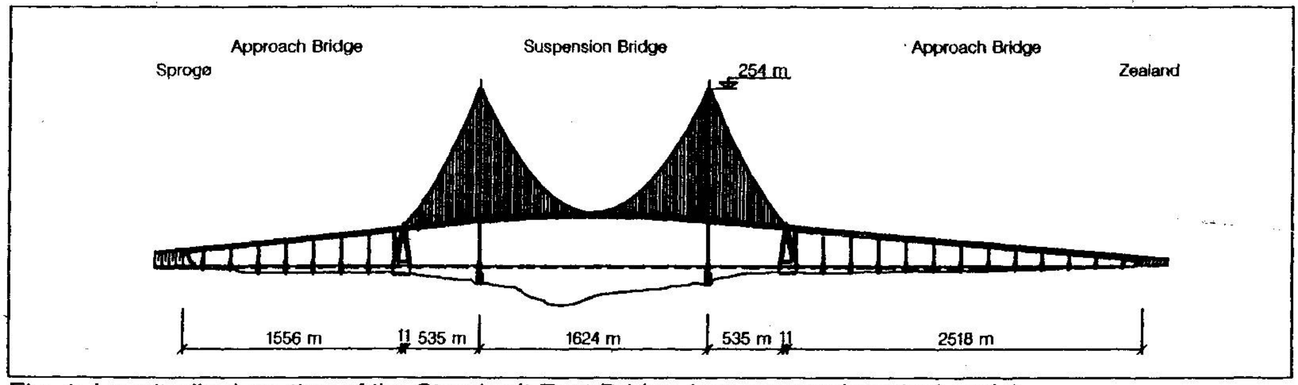


Fig. 1: Longitudinal section of the Storebælt East Bridge (exaggerated vertical scale)

The water depth in the deep part of the channel reaches more than 50 m, but at the positions of the pylons it is reduced considerably to about 21 m. At the anchor blocks and the approach span piers the maximum water depth is less than 12 m.

Generally, the geology in the bridge alignment consists of clay till on top of a marl layer and below that limestone. The clay till has a thickness of 20 to 30 m except at the pylons, where it is reduced to 8-10m . In the deep channel the till formation is missing.

Throughout the initial project phases, it was anticipated that the structures would be founded directly on this upper pre-consolidated clay till strata after excavating a thin top layer of unsuitable late or post glacial deposits.

2 PYLON FOUNDATIONS

As in situ casting of the pylon foundations within de-watered construction pits would obviously not be feasible due to the large water depth and the nearby navigation channel, a construction method was conceived based on placing pre-fabricated concrete caissons on compacted crushed stone beds. Initially these stone beds were assumed to be 1.5 m thick, but as the detailed soil investigations showed considerably lower and more variable strength and deformation parameters of the 8-10 m layer of clay till than expected, it was decided to excavate the clay till and place 5m thick stone beds on top of the stronger and more uniform marl strata.

2.1 Foundation Stone Beds

The stone material used is well-graded 5-90 mm crushed hyperite quarried from Kragerø, Norway. Compaction of the stone beds to the specified 96% of the maximum dry density was performed in two layers of approximately 2m thickness each, leaving a top layer of about 0.4 m which was loosely placed and screeded to provide a levelled surface for placing the caissons. The vibrator plates used for the compaction measured 25 m^2 weighing 75 tonnes. As it was not possible to device proper methods for in situ compaction control, a prescription system was adopted verified by full-scale underwater compaction trials in a test pit established onshore. Large scale triaxial tests on material excavated from the test pit after compaction showed very high values of the triaxial secant angle of friction , phi>50°. In all the tests the material dilated at failure, but the rate of dilatancy was lower than expected. For further information see Steenfelt and Foged [1].

2.2 Foundation Caissons

The foundation caissons, each covering an area of 2,770 m^2 , are 78 m long and 35 m wide, see Fig. 2. However, the corners and the edges are rounded in order to reduce the water blocking effect and to limit damage to possible colliding ships by avoiding sharp corners and edges. They are divided into 60 cells by internal walls and are 20 m high, including a 3 m thick plinth cast in situ on top of the cellular caissons within a temporary steel cofferdam. This was required because the top of the caisson is kept 3.5 m below water level in order to avoid the undesirable visual effect of the slender pylon legs and the voluminous caisson standing one on top of the other.

The bottom slab of the caissons has a variable thickness between 0.95 m and 1.1 m, creating a roof-shaped underside to the slab. Around the periphery of the slab and below some of the internal walls 0.5 m deep skirts are provided, designed to penetrate about 0.3 m into the uncompacted screeded top layer of the stone beds, see Figure 2. The voids thus formed between the roof-shaped underside of the slab and the stone bed were subsequently filled with sand/cement grout through pre-installed pipes, the skirts serving to confine the grout. For each of the caissons the quantity of grout was some 725 m^3, placed in about 40 hours applying a pressure of 8 bar at the inlet pipes.

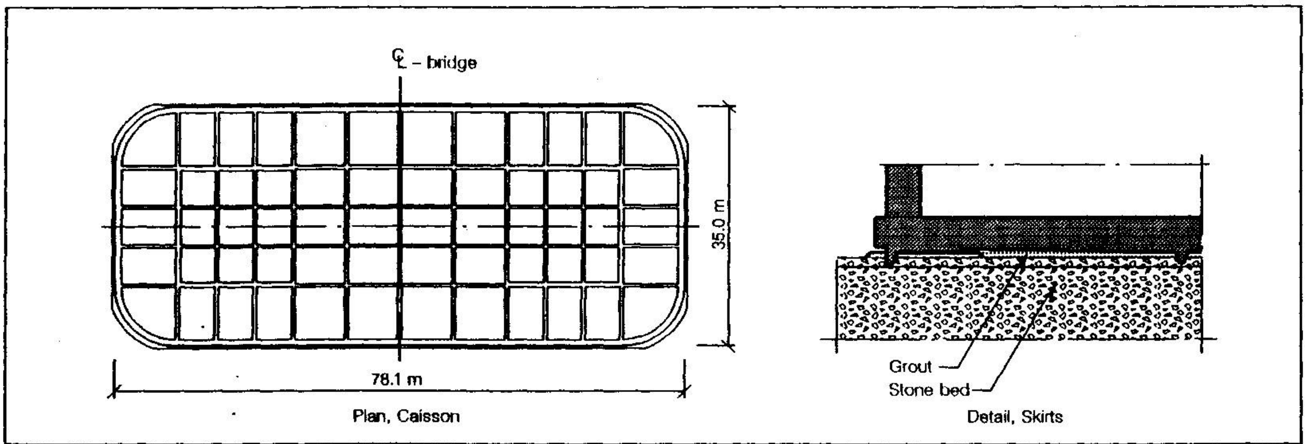


Fig. 2: Plan and Detail of Foundation Caisson

2.3 Design for Ship Collision

Due to the large water depth at the pylon locations, it was not feasible to arrange protective artificial islands around the foundation caissons as this would restrict the free opening for the navigation route too much. Therefore, the pylon foundation had to be designed to resist the impact from a fully loaded 250,000 DWT tanker travelling at a speed of 10 knots (5.1 m/s). This design vessel was selected being the maximum size which can pass the bridge considering the draft limitation of 17.5 m for the navigation route from the Baltic Sea.

The load-time curve for the impact against the non-moving pier was established based on theoretical considerations of the energy dissipated by adding up the contributions during plastic deformation of all basic structural elements of the ships hull. The procedure developed for bow crushing analysis is based on an upper-bound plasticity theory which can also handle rigid body motions of crushed and non-crushed parts of the structure. Validation of the numerically predicted crushing loads has been done by comparison with experimental crushing results of small scale bow models.

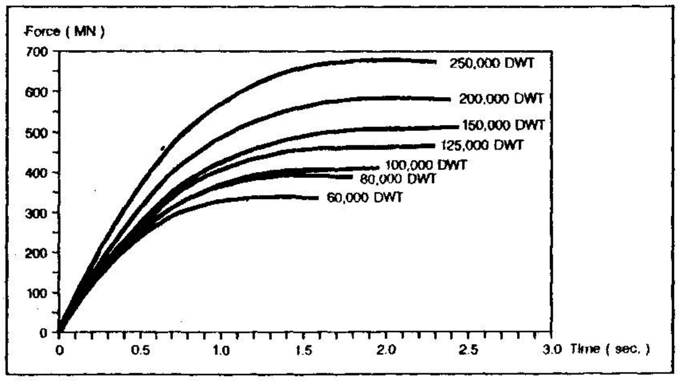


Fig. 3: Idealised load-time curves for ship impacts

In order to resist an impact load of 673 MN without permanent movement a very large effective vertical load on the foundation area of around 2,000 MN is required. This has been achieved partly by filling each caisson with 37,500 m^3 of hydraulically pumped sand being the cheapest weight material available. In this respect the inherently large dead weight of the concrete pylon itself -around 950 MN- is an advantage compared to a steel pylon solution, the lower weight of which would have had to be compensated by a considerably larger caisson with increased ballast material weight.

3 ANCHOR BLOCKS

Most existing major suspension bridges have their anchorages on land in rock formations, which makes the design fairly easy. However, for the East Bridge the challenge for the designers was much greater because the anchorages -subjected to a horizontal force of about 600 MN- had to be constructed on clay more than 2 km from the coast at water depths of approximately 12 m.

It was clear from the beginning that gravity-type anchorage structures would have to be adopted, constructed either by placing pre-fabricated caissons on stone beds or by casting the structures in situ within huge de-watered construction pits. Both concepts were developed for tendering and closely equally priced by the contractors at tendering. However, the pre-fabricated caisson solution was selected.

3.1 Foundation Stone Beds

The soil conditions at the anchor block locations consist of 20 m of clay till on top of a 40 m thick layer of marl. Excavating the clay till under water would disturb its surface and the anchor block could slide along this thin disturbed zone. Comprehensive large-scale field tests and studies to overcome this problem were carried out in the initial project phases. Different solutions for penetrating the disturbed zone by short steel sheet pile walls or large diameter steel piles were proposed as well as several configurations of wedge shaped stone beds. Based on these studies a solution with two individual wedge shaped stone beds without support of the middle section of the caisson was selected for tendering, see Fig. 4. This option was considered to give more well-defined contact pressures at the two separate wedges as compared to solutions with contact over the entire base area. For further discussion of this aspect, see Mortensen [2]. In addition a solution with large diameter steel dowels was included as a variant in the tendering, but this was priced considerable higher than the basic stone bed solution. The stone materials used as well as method for compaction, underbase grouting etc. are the same as described for the pylon foundations.

3.2 Foundation Caissons

The foundation caissons, each covering an area of 6,100 m^2, are 121.5 m long and 54.5 m wide and divided in three parts. Only the front and rear parts are in contact with the supporting stone wedges as explained above. The height of the pre-fabricated part of the caissons is 15 m having a weight of 50,000 tonnes when towed to the bridge site whereas the remaining parts of the anchor blocks above level +3.0 m were cast in-situ.

The rear part of the caissons contains the anchorage massifs comprising 18,000 m^3 of concrete which was cast in-situ partly inside some of the cells in the pre-fabricated caisson. Other cells in the rear part were ballasted with olivine (heavy sand) and iron ore.

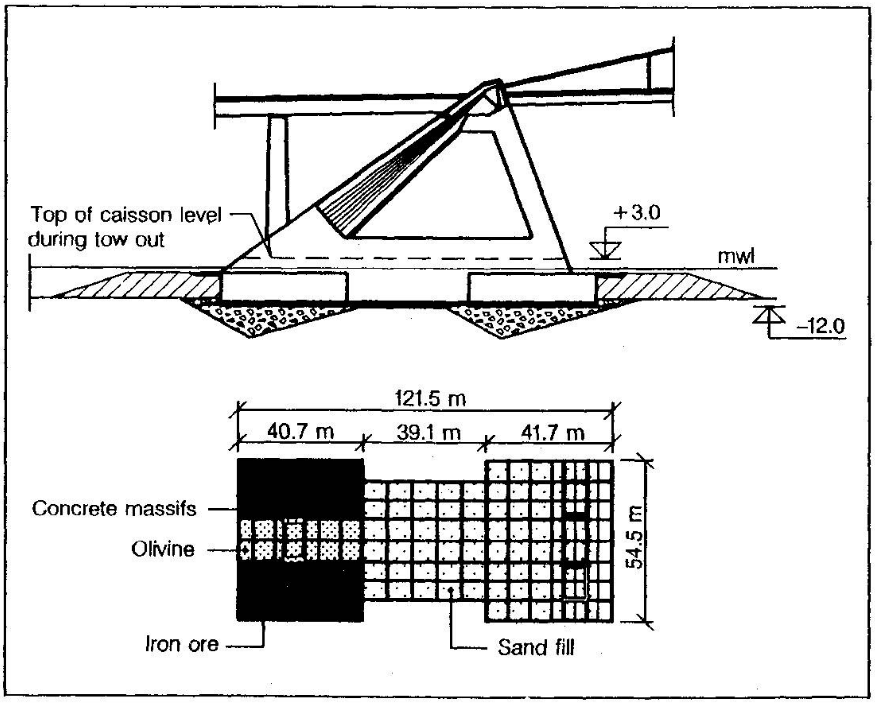


Fig. 4: Elevation and Plan of Anchor Block Caisson.

The middle part of the caisson, which has no contact with the underlying ground, is partly sand-filled; the longitudinal walls are post-tensioned with vertical bars; and the bottom slab is heavily post-tensioned longitudinally with a total force of 363 MN. The front part of the caisson is filled with hydraulically pumped sand.

The anchor block caissons are protected against ship impacts by artificial islands constructed from soil materials dredged nearby. This dredging -known as compensation dredging- was done in order to open up for increased water flow to compensate for the blocking effect of the bridge substructures. The protection islands are elliptical shaped with the long axis in the direction of the predominant water flow in order to reduce the blocking effect.

3.3 Structural Analyses

The structural calculations for the anchor blocks were mainly based on a linear elastic finite element model using the IBDAS programme, whilst the geotechnical calculations and the determination of soil pressure distributions were carried out by separate analyses models comprising use of the non-linear finite element program ABAQUS. For further information see [3]. The soil pressure distributions were then applied on the IBDAS model.

Due to the unique concept of the anchor blocks involving application of heavy post-tensioning and ballast material in various construction phases as well as the construction of the inclined upper elements by free cantilevering from the caisson deck, it was necessary to analyse a substantial number of construction phases and keep track of the built-in stresses and their re-distribution due to creep and shrinkage in subsequent phases up to the final condition. The main construction phases included:

- Construction of the caisson in the dry dock including tensioning of tendons in the bottom slab and vertical bars in the longitudinal walls of the mid-part.
- Tow-out and installation of the caisson including effects of ballasting for trimming and wave loads.
- Ballasting of the caisson with olivine/iron ore and sand before construction of the upper inclined elements.
- Construction of the upper inclined elements by free cantilevering.
- Final condition after connection of the inclined elements by the top cross beam.

The calculations showed that the stresses built-in during the construction phases were very important and required substantial strengthening in particular for the lower part of the caisson structure. However, the model was very useful for optimisation of the entire construction process including adjustment of post-tensioning levels and ballasting procedures at various stages.

4 APPROACH BRIDGE PIERS

The decisive criterion for the design of the approach bridge piers is ship impact loads, which was established based on a comprehensive collision risk model. The model estimates the risk of collapse as the sum of four risk scenarios, each representing a certain ship track pattern categorised as follows:

- Ships following the ordinary, direct route at normal speed. Accidents mainly due to human error or unexpected problems with propulsion/steering system when approaching the bridge.
- Ships failing to change course at the turning points of the navigation route
- Ships taking evasive action for other vessels when approaching the bridge.
- All other track patterns, e.g. off-course ships and drifting ships.

Based on the collision risk model the design basis for the ship impact loads was established, see Fig. 5.

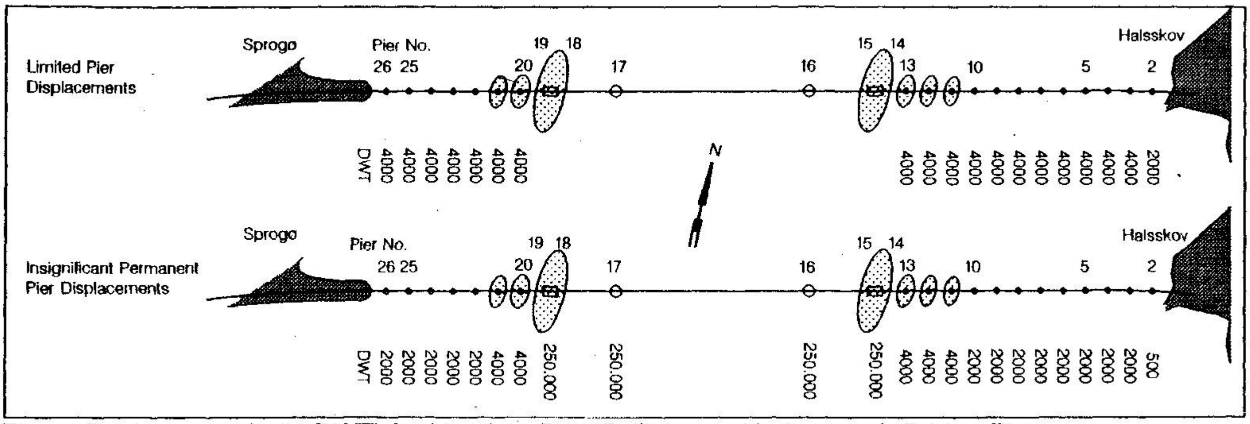
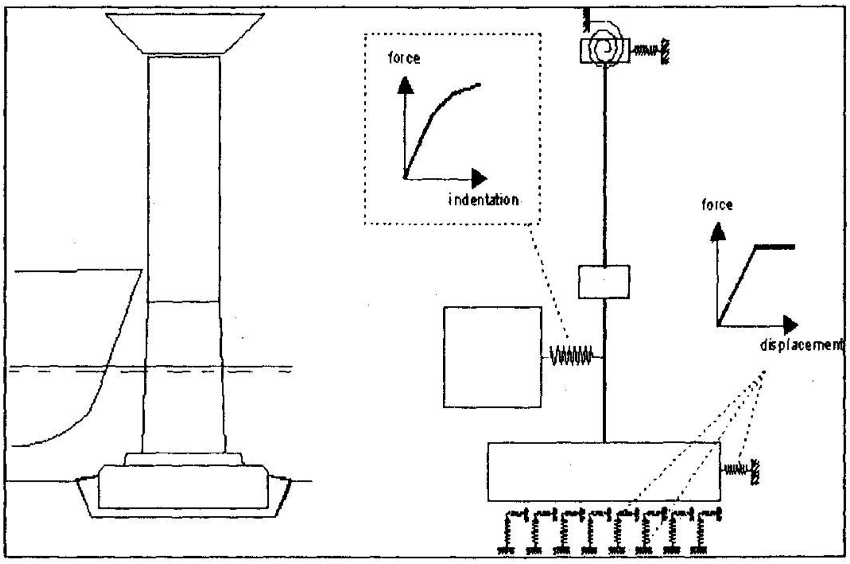


Fig. 5: Design vessels (in DWT) for Head on Bow Collisions with Approach Bridge Piers

In order to achieve the same overall base dimensions, 23 m x 19 m, of all the approach bridge piers, it was decided to construct artificial protection islands around some of the piers close to the navigation channel, see Fig. 5. The remaining piers were designed to resist an impact load of 69 MN from a 4,000 DWT ship allowing limited displacements at the pier top of 200 mm and 500 mm in the transverse and longitudinal directions respectively. These displacements will require repairs, however, without closing the bridge for more than about one month. In addition, the piers shall be able to resist an impact load of 46 MN from a 2,000 DWT ship without insignificant permanent displacements which will not require closing for traffic.



A simplified but still advanced computer model was developed for the ship impact analyses, see Fig. 6. The model comprises: a non-linear elasto-plastic model of the soil-structure interaction, a stiff foundation caisson, elastic beam elements for the pier shaft and a concentrated mass and spring at the pier top representing the bridge girder. To verify the simplified model, independent 2D finite element analyses were carried out using two different programs ABAQUS and FENRIS. In addition the effects of eccentric impacts were studied by 3D models with a more refined modelling of the soil, including local week areas etc. in FENRIS.

Fig. 6: Idealised model of the Ship Collision.

The comparison of results from the simplified model and the finite element analyses showed a good agreement with differences in ultimate bearing capacity being some 10% only. However, the finite element models give a better description of the behaviour of the soil especially when failure is dominated by overturning and twisting moments. For further information, see Feld and Gravgaard, [4].

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Present situations and problems in design and construction of caisson foundations in Japan

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Summary: The caisson foundation is a one of the typical foundation type in Japan, and mainly used for larger-scale bridge foundation because it's big bearing capacity. However, recently, the share of caisson foundation has been decreased. The caisson of this situation can be considered that the difficulties of construction.
Therefore, various technical development has been carried out and some of them are used in-site, for example, the unmanned excavation system aided by remote-controlled excavation machines and so on.
The seismic design of bridge foundation including caisson foundation in Japan revised several times based on the lesson from large earthquake, for example, Niigata Earthquake (1969), Hyougoken-nanbu Earthquake (1995). Present seismic design method has been systematized with the analysis of a seismic response.
This paper shows the present situations and problems in design and construction of caisson foundation under such circumstance in Japan.

1. Introduction

This report describes the pneumatic caisson method and the open caisson method as a deep foundation. The former method is performed using a caisson including a worker chamber made up of external walls and a ceiling slab at its bottom, forming to be open to the ground that is excavated beneath it. As compressed air is pumped into this working chamber to keep it free from water, and workers and machines can excavate and remove soil from beneath it, the caisson body is lowered until the excavation has been completed down to the design bearing stratum.

Its features are:
[1] Because the excavation work can be done in an atmosphere identical to that at the ground surface, it is able to excavate all kinds of soil from clay to rock.
[2] This means that it is possible to count on the excavation work being completed satisfactorily and to execute the work according to the construction project plan.

Shortcomings are:
[1] It is difficult to construct extremely deep foundations.
[2] The excavation facility is large scale.

The latter method is a performed by lowering a caisson body to the design bearing stratum as clam shell buckets or grab buckets are used to excavate and remove the soil inside the caisson.

Its features are:
[1] It is performed by repeating a simple execution procedure.
[2] Depending on the soil quality, it can be used to construct extremely deep foundations.

Shortcomings are:
[1] Excavation under cutting edge cannot be performed.
[2] It is difficult to deal with settlement or inclination of the caisson body.

Table.1, which presents the kinds of foundations selected for Japanese highway bridges (span length of 20 m or more) up to 1995, reveals that the number of caisson foundations constructed has declined in recent years.

	1966	1975	1985	1995
Spread Foundation	125	1.263	1.526	1.869
Pile Foundation	207	1.383	1.996	2.942
Caisson Foundation	117	170	145	81
Steel Pipe Sheet Pipe Foundation	–	–	30	59
Cast-in-site Diaphragm Wall	–	–	–	6
Others	–	–	–	38
Totals	449	2.816	3.697	4.995

Table.1 Changing Highway Brige Foundation Construction Methods in Japan

There are two reasons for this tendency.
[1] The fact that pile foundation has high applicability for various kind of soil and it is now possible to construct large diameter piles.
[2] The development of new forms of foundations stood for steel pipe sheet pile foundations and diaphragm walls foundation.

Fig. 1 shows that many of those caisson foundations, which support long span bridges, have been constructed at locations with very deep water. The reason of this situation is that they are appropriate for deep foundations thanks to the benefits cited above and the large bearing capacity of a caisson foundation.

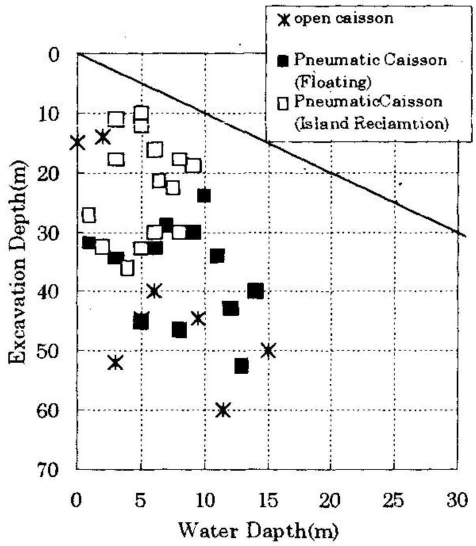

Figure.1 Underwater caisson Foundation Executions in Japan

2. History of the Development of Caisson Execution Technology in Japan

2-1. Pneumatic Caisson

The first pneumatic caisson as highway bridge foundation constructed in Japan was the foundation of the Eitai Bridge (Tokyo) in 1925. Afterwards this method was frequently used to construct deep foundations until the late 1960s. It is necessary to limit the period that workers spend in the working chamber in order to protect them from decompression illness and other dangers of working in high pressure environments, therefore the higher the pressure in the working chamber, the shorter the working time that excavation can be performed. As a result, it was necessary to replace manual excavation methods with more efficient mechanized excavation methods.
In the 1960s, tracked bulldozer-shovels powered by electric motors were developed. But because drivability of this machines were very bad at the excavation of soft clay, new systems with the excavation shovels moved along rails installed on the ceiling of the working chamber were developed during the 1970s. Because these new systems were operated by workers seated on the machinery itself, they were still subject to the effects of the highly compressed air. In 1989, a remote operating machine was developed. Operator can drive this machine from operating rooms located at ground level and free from the high pressure atmosphere in the working chamber.

2-2. Open Caisson

The first open caisson as highway bridge foundation must be the bridge that was completed in 1887. Afterwards many reinforced concrete caissons were constructed. The excavation was done primarily using the clam shell bucket method, and as the assistant method of caisson settlement, pressurized water or compressed air were injected towards the ground from the external wall. Then in the 1970s, a method of applying a special friction reduction sheet to the external wall was developed, and in the late 1980s, a press-in method employing ground anchors and center hole jacks was developed. These innovations are still in use.

3. Recent Trends in Caisson Execution Technology

3-1. Pneumatic Caisson

The remote operation method is often used when the pressure in the working chamber exceeds 0.294 MPa, and it had been used a total of 46 cases according to a May 1997 survey. Even when this method is employed, workers have to work in the high-pressure area to inspect and repair the machinery, lighting fixtures, and so on. When workers breathe air at a pressure in excess of 0.392 MPa, they suffer the symptoms of nitrogen narcosis. To prevent this from happening, a respiration system that supplies a helium, nitrogen, and oxygen mixture with low nitrogen content was introduced in 1995 and has now been employed at five caisson foundation construction sites. The method is now applied where work is performed in high-pressure conditions experienced at a depth of 70 meters under the surface of the water.

Category		Location	Bridge Name	Foundation Specifications (m)		Super-structure	Center Span Etc. (m)	Water Depth (m)	Vertical Ground reaction per unit area at the bottom of foundation (kN/m²)	Foundation Ground
				Plane Dimensions	Embedding Depth from Sea Bed or Ground Surface					
Underwater Foundation	Floating caisson	Tokyo Bay / harbor	Rainbow Bridge	70.0×45.0	34.5	Suspension bridge	570	12.0	1290～450	Alluvial clay to hard pan
	Double wall cofferdam	Osaka Bay / harbor	Kodai Bridge	40.0×40.0	30.5	Gelbar truss bridge	510	4.0	420.0	Alluvial clay to diluvial gravel to rock
	Island reclamation	Hokkaido / harbor	Hakucho Ohashi Bridge	46.0×33.0	21.0	suspension bridge	720	7.0	–	Alluvial clay to diluvial gravel
	Floating caisson	Osaka Bay / harbor	Higashi Kobe Bridge	35.0×32.0	27.5	steel cable-stayed bridge	485	9.0	656～453	Alluvial clay to diluvial gravel
	Floating caisson	Ise Bay / harbor	Meiko Chuo Bridge	34.0×30.0	35.0	steel cable-stayed bridge	590	14.0	910.0	Alluvial clay to diluvial gravel to rock
	Floating caisson	Ise Bay / harbor	Meiko Nishi Bridge	40.0×25.0	28.0	steel cable-stayed bridge	405	12.0	–	Alluvial clay to diluvial gravel
	Floating caisson	Korea / Inchon River	Eiso Bridge	47.0×18.0	30.85	Suspension bridge	300	10.0	–	–
	Constructed on site	Ise Bay / river mouth	Inde River Bridge	26.0×10.0	40.0	Steel box girber bridge	97.5	2.5	–	–
On-land Foundation		Tokyo Bay / harbor	Rainbow Bridge	70.0×45.0	39.0	Suspension bridge	570	–	1098～341	Alluvial clay to hard pan
		Kan-etsu Expressway / mountains	Nagai River Bridge	47.6×18.0	26.0	PC box girber bridge	123	–	–	Gravel
		Tomei reconstruction / mountains	Tomei Ashigara Bridge	ϕ18.0	22.0	PC cable-stayed bridge	185	–	–	To rock

Table. 2 Large scale Pneumatic caisson Foundations

The following unresolved challenges remain.

[1] Shortening the caisson body construction work process to balance it with the excavation process

[2] Gaining the ability to work at deeper levels by researching saturation methods for mixed gas respiration and the joint use of other construction method such as the soil freezing methods.

3-2. Open Caisson

The Super Open Caisson System (SOCS method), an automatic open caisson technology, has been developed as a joint public - private sector research project at the Public Works Research Institute of the Ministry of Construction.

This new technology provides the following benefits.

[1] It includes newly developed excavation machinery that can reliably excavate the ground under the cutting edge of the caisson down to great depths. It permits automatic operation of the excavation machinery in harmony with soil lifting equipment.

[2] It permits the excavation and caisson attitude data to be linked to perform automatic press-in settlement control.
[3] To rationalize the execution of the caisson body construction, prefabricated caisson bodies were developed.

This new system has been used to perform excavations down to a maximum depth of 53.5 m. All that remains to be done is to loser its cost and expand the range of soil types it can be used to excavate.

Category		Location	Bridge Name	Foundation Specifications (m)		Super-structure	Center Span Etc. (m)	Water Depth (m)	Foundation Ground
				Plane Dimensions	Embedding Depth from Sea Bed or Ground Surface				
Underwater Foundation	Crane lowered	Hiroshima Bay / harbor	Umeda Bridge	ϕ16.0	50.0	Steel box girder bridge	250	10~15	Alluvial clay to diluvial gravel
	caisson	Hiroshima Bay / harbor	Hiroshima Bridge	ϕ10.0	45.0	Steel box girder bridge	150	15.0	Alluvial clay to diluvial gravel
	On-land execution	Ise Bay / river	Kiso-sankyo Bridge	ϕ11.0	52.0	Steel Truss	–	–	Sand/clay to gravel
	(Island reclamation method)	Ariakeumi / river	Nitta Bridge	ϕ9.0	40.5	Stiffened arch bridge	–	9.0	Alluvial clay to gravel
On-land Foundation		Kanetsu Expressway / mountains	Nagai River Bridge	43.5×16.5	10.0	PC box girder bridge	123	–	To gravel

Table. 3 Large Scale Open caisson Foundations

4. Caisson Foundation Design Methods: Present Status and Future Challenges

4-1. Present Design Methods

4-1-1. General Items
Design standards for highway bridge have been issued as a notification of the Ministry of Construction under the title, "Technical Standards for Bridges and Highway Viaducts." Regarding standards for caisson foundations, since "Design of Caisson Foundations" was issued in 1970, it has been steadily revised and enacted in 1980 under the title, "Specifications for Highway Bridges, Part IV: Substructure " as integrated guidelines for substructure. The seismic design method for foundations stipulated in these standards were the seismic coefficient method. By the lesson from the Hyogo-ken Nanbu Earthquake of January 1995, seismic design was radically revised; the seismic coefficient method was supplemented by verification based on the ductility design method.

4-1-2. Design Model of a Caisson Foundation
a) Rigidity of the caisson body
A caisson body is modeled as a single column, and is in principle, an elastic body. But the ductility design method accounts for a decline in rigidity caused by cracking or the yield of rebar.
b) Ground Resistance
The ground resistance is an elasto-plastic spring model as shown in Figure. 2 to account for the horizontal resistance, the vertical shear resistance on the front, the horizontal shear resistance on the side, the vertical resistance, and the shear resistance on the bottom. But accounting for the execution procedure, the self-weight was assumed to act only on the bottom of the foundation. The maximum value of the horizontal resistance is treated as Coulomb's passive resistance earth pressure, and the ductility design method accounted for three dimensional expansion of the passive resistance range.

4-1-3. Caisson Foundation Stability Calculations
a) Normal conditions, earthquake conditions in case of seismic coefficient method and storm

conditions
On the premise that the caisson body and the ground at the bottom surface are in the elastic range, it is verified that the horizontal displacement, vertical and horizontal ground resistance at the bottom, and the member stress are all below the allowable values.

b) Seismic design based on the ductility design method
This method verifies that the horizontal capacity of the foundation is equal to or greater than the ultimate horizontal capacity of the bridge pier body, or in other words that when a load that corresponds to the ultimate horizontal capacity of a bridge pier body acts on foundation, the overall behavior of the foundation does not reach the yield of the foundation. When the term "yield of the foundation" is defined as the state where horizontal displacement at the inertial force action point of the superstructure begins to rise rapidly as the horizontal load rises.

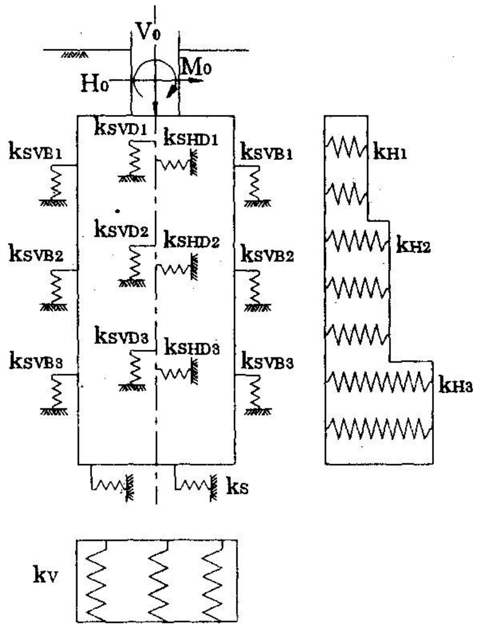

Figure. 2 Ground Resistance

4-2. Bearing Capacity Design: Present Status and Challenges

Because one feature of the caisson construction method is that it is possible to confidently embed a body with high rigidity on the bearing stratum in order to obtain high bearing capacity, this section describes the evaluation of bearing capacity.

4-2-1. Present status of bearing capacity design
The ultimate bearing capacity of the ground at the bottom of a caisson foundation for a highway bridge is generally found based on the results of soil tests and soil explorations and using a formula (1) that is similar to the Terzaghi bearing capacity formula premised on the general shear failure of the ground. Because a deep foundation such as a caisson foundation resists the horizontal load or overturning moment aided by the ground at its front surface, only a little of the load is borne by the ground at its bottom surface. So equation (1) ignores the eccentricity and inclination of the load.

$$qd = \alpha \cdot c \cdot N_c + 0.5\ \beta \cdot B \cdot \gamma \cdot N_\gamma + q \cdot N_q \quad \text{-------------------(1)}$$

Where:

qd	=	ultimate bearing capacity (kN/m^2)
α , β	=	shape factors
N_c, N_γ , N_q	=	bearing capacity factors
c	=	cohesion of the ground at the bottom surface (kN/m^2)
B	=	width of foundation (m)
γ	=	unit weight of the bottom surface ground (kN/m^3)
q	=	weight of the soil above the foundation bottom (kN/m^2)

When a caisson foundation is deeply embedded, the ultimate bearing capacity calculated using formula (1) is extremely large. But the quantity of settlement during this ultimate bearing capacity is not clearly known. In the case of large foundation dimensions where it is impossible to ignore the quantity of settlement for the stability calculations, a designing maximum value of the allowable vertical bearing capacity is established considering this fact in order to perform the stability calculations for normal condition or the seismic coefficient method referred to above. This maximum value, which is shown in Figure. 3, is obtained by modifying the results of plate loading testing of the pneumatic caisson added an engineering judgment.
To use the ductility design method, an elasto-plastic spring model with the ultimate bearing capacity of the bottom surface ground obtained from formula (1) as the maximum value is established.

4-2-2. Challenges

The plate-loading test performed inside the working chamber of a pneumatic caisson during construction has, in some cases, obtained an ultimate bearing capacity of about 10,000 kN/m^2. But the experimental maximum value of the bearing capacity shown in Figure. 3 is set very low to obtain a value on the safe side in engineering terms. It is, therefore, necessary to establish more precise formulae to estimate bearing capacity and quantity of settlement in order to perform more economical foundation design work.

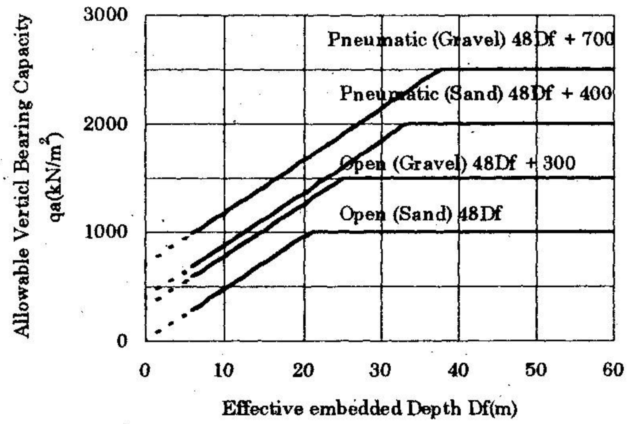

Figure. 3 Maximum Value (Normal Conditions) of the Allowed Vertical Bearing Capacity of the Ground at the Bottom Surface of a Caisson Foundation

It has been pointed out that there are discrepancies between the measured and theoretical equation (1) values with the static formula, even in the case of a shallow foundation: the existence of the scale effect of the bearing capacity factor for example. The scale effect of the bearing capacity factor is gradually being clarified thanks to the performance in recent years of more precise gravity field or centrifuge model tests, and by performing large bearing capacity tests of spread foundations in natural solid ground[1].

And to expand and apply the shallow foundation bearing capacity theory to deep foundations, various problems including that of the disparity of the failure mechanisms of the two kinds of foundation must be overcome. For example, while the embedment effect increases the bearing capacity, it also makes the stress-dependency of the shear resistance angle of earth ϕ used to calculate the bearing capacity factor becomes remarkable. It is also not clear if the shape factor of a shallow foundation that is set primarily based on test results and experience-based judgments can be applied to a deep foundation without modification. And even the deep foundation theory proposed by Meyerhof and Vesic is still plagued by many unresolved problems that are currently under research. These include the problem of the discrepancies on the theoretical model caused by the difference in the size of piles and caissons, the determination of the ground constants for design use, and so on. Remarkable progress in computation technology and ground exploration technology seen in recent years are expected to contribute to the final establishment of a bearing capacity estimation formula for caisson foundations based on the perfect plasticity theory and elasto-plastic theory.

On the other side, there is lively research activity underway to develop methods of estimating bearing capacity using FEM analysis methods that precisely model the strength deformation properties of ground based on the results of extremely detailed soil tests. In order to obtain data needed to verify the effectiveness of this method, eccentric loading test of spread foundations incorporating pneumatic caissons are being conducted on dense fine sand ground during foundation construction [2]; Through these tests and analysis that is expected to provide a practical working bearing capacity estimation method that also accounts for settlement so that it can be applied to deep foundations as well as shallow foundations.

Reference:

1) Kusakabe, O., Maeda, Y., and Ohuchi, M.: Large-scale loading tests of shallow footings in pneumatic caisson, J. Geotech Engrg, ASCE, Vol. 118, No. 11, pp. 1681 - 1695, 1992.
2) Oyake, T., Ohuchi, M., Kimura, Y., Nakano, M. and Tatsuoka, F.: Field model bearing capacity tests on a Pleistocene sand deposit, Foundation Failures, May 1997, Singapore, pp 1 - 12.

PROBLEMS OF CONSTRUCTION OF CAISSON FOUNDATIONS OF THE SECOND HOOGHLY BRIDGE (VIDYASAGAR SETU) AT CALCUTTA AND THEIR SOLUTIONS

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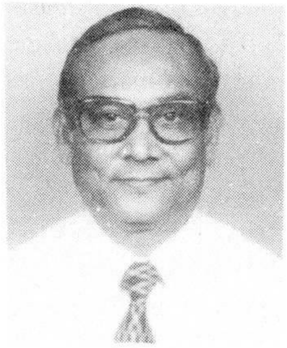

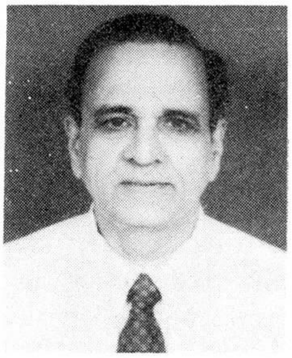

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SUMMARY:

The 822.96 m long cable stayed bridge, over river Hooghly at Calcutta, with a central span of 457.2 m and named "***Vidyasagar Setu***", is one of the most outstanding long span bridges not only in India but also in the world. Vidyasagar Setu was planned and constructed to relieve the extraordinary congestion of traffic over the other old existing long span bridge, nearby, popularly known as the ***"Howrah Bridge"***.

This new bridge has two carriageways of three lanes each separated by a median verge and has a clearance of 30.0 m above the HTL to allow passage of oceangoing vessels. Each of the four foundations of the bridge, comprise of twin circular caissons. The location of the foundations and the caisson diameters are as below:

F-1 comprising of twin caissons on Calcutta bank	:	12 m
F-2 comprising of twin caissons on Calcutta bank	:	20 m
F-3 comprising of twin caissons on Howrah side but inside the river	:	23 m
F-4 comprising of twin caissons on Howrah side and inside the river	:	8 m

In the background of influencing factors e.g. high sensitivity of river regime, navigational traffic, the alluvial nature of the bed, high expected long term settlements, and the need to minimise them, several challenging problems of complicated nature were encountered in design and construction particularly for foundations F-2 & F-3. The paper describes the various techniques of construction adopted to meet the challenges particularly those connected to the tower foundations F-2 & F3 and touches upon the problems related to the anchor foundations F-1 & F-4, the design concepts and the methodologies applied, to overcome the problems, that finally enabled the emergence of this aesthetically pleasing bridge.

1.0 LAYOUT

The general elevation and plan of the bridge is shown in fig.1 and the layout of F-3 and sections in fig.2. Layout of F-2 is similar to F-3, but the length of the pier in plan is slightly lesser than the pier over F-3. Layout of F-4 is shown in fig.3. F-1 is similar to F-4 but with larger sizes. Photograph 1 shows a view of the completed bridge.

2.0 THE PROBLEMS FOR CAISSONS UNDER F-2 & F3 AND THE SOLUTIONS.

2.1 Problems connected to the construction of the caisson plugs

Originally it was required that the bottom plugging be carried out in the dry. This could have been achieved by plugging under pneumatic conditions only, if to be done in one operation. However to obviate risks and dangers associated with plugging under pneumatic condition under a large head of water, pneumatic plugging was ruled out, as a measure of prudence and abundant caution.

2.2 The solutions and methodology finally adopted to overcome the problem

On examining other alternatives, a safer and satisfactory alternative of forming the plug in two stages, was adopted. In this methodology an under water plug in colcrete/concrete to sustain base pressures due to self weight of caisson and the vertical pressure caused under the plug by a hydrostatic head reckoned from HTL or LTL, after placing vertical steel bond bars in this plug for eventual integration, was cast in the first stage. After curing followed by dewatering, a layer of RCC plug over the first stage plug was cast in dry and cured to integrate with the first stage plug. Both these plugs were individually and integrally designed to sustain the various stage loadings that would be incident on them including the service loading. Fig.4 shows the typical integrated plug.

2.3 The problem of casting the caisson caps for F-2 and F-3

To cast the heavy capping slabs, an elaborate and supporting system would normally have been required, even though constructing such a structure in dry was possible. However this would have been uneconomical and time consuming.

2.4 The solutions and methodology of construction

Precast concrete shuttering slabs were designed to sustain the self weight of the caisson cap. Connector reinforcements were provided to integrate them monolithically with the caisson cap which was designed to cater for loadings that would occur under service. These precast slabs being light, in actual construction, handling was not difficult and the system worked well and successfully during actual construction.

3.0 PROBLEMS OF CONSTRUCTION OF CAISSONS FOR F-3

3.1 The problem

The river is sensitive to silting and scouring even with slight disturbances in regime conditions. To obviate destabilisation of the bed, large obstructions to the river flow was considered undesirable. Construction of caissons with the aid of cofferdams was therefore ruled out.

3.2 The solution and methodology of construction

The alternative of constructing by the floating caisson method was adopted. The depth of water at the foundation location was about 15.0 m during high tides. Structural steel caissons 22 m high and weighing about 800t were used. Initially an 8.0 m high steel caissons weighing about 360t were assembled over a specially designed tilting slipway platform (photograph 2), launched into the river during the high tides and transported to location by tugs. The steel caissons were aligned by using very high precision theodolites and distance measuring units (distomats) from two survey stations on opposite river banks and one survey station constructed in the river on the upstream side of the location. The caissons were grounded at the location using a combination of concrete and water ballast providing the flexibility to refloat the caisson in an eventuality. The design and construction of the steel caissons, tilting slipway platform, floating operation, anchoring arrangement, towing operation, alignment and grounding and sinking operations, including many other associated activities needed a very detailed investigation and meticulous planning to achieve the desired results, which was very successfully done at site. Photograph 3 shows caissons of F-3 under construction.

3.3 The problem of constructing the pier over F-3

As mentioned above, obstruction to water way had to be minimum. Therefore the top level of the caisson was kept very low at +0.609 KODS i.e. 6.76 m below high tide level. Construction of caisson caps and part of the pier system had therefore to be carried out underwater, throwing a serious challenge.

3.4 The solution and methodology adopted

A circular RCC wall of short height was constructed over the outer wall of the twin caissons followed by the construction of 7.0 m high dismantlable circular steel cofferdams. The caisson caps and a part of the pier box was then cast within the circular cofferdam upto a height slightly above the HTL. Four temporary RCC columns were cast alongside the partly cast pier boxes to support steel trusses spanning over the twin caissons. Thereafter a 6.0 m wide & 16.0 m long portion of the connecting pier beam weighing 600t approx., in the shape of a trough and being a part of the pier system, was cast above the high tide level. The soffit shuttering for this pier beam was supported by a support system suspended from the truss. A portion of the circular steel cofferdams was dismantled and the pier beam partly cast with the truss support, was lowered into position by means of hydraulic jacks and seated over brackets provided at the soffit level of the caisson caps. Vertical steel gates were then erected on the outer faces of the pier walls at the junctions between the connecting pier beam and the pier box. These gates were sealed with the help of divers. Thereafter the pier box with the pier beam was dewatered completely, which resulted in a very effective sealing by water pressure from outside. The junction at the soffit slab level and between the pier walls were integrated by insitu concrete. The joints were shaped in the form of saw-tooth type castellations with connector reinforcements for proper shear transfer. The steel cofferdams were then completely dismantled and the rest of the pier box above the HTL and the pier cap constructed in the conventional manner. A difficult part in this construction was to achieve the enmeshing of the horizontal bars of the pier boxes and the connecting beam, and the vertical bars of the steining. Hence every space between the steining bars was actually measured and a layout drawing made. The horizontal bars in the pier beam were slightly adjusted before lowering to enable a smooth passage.

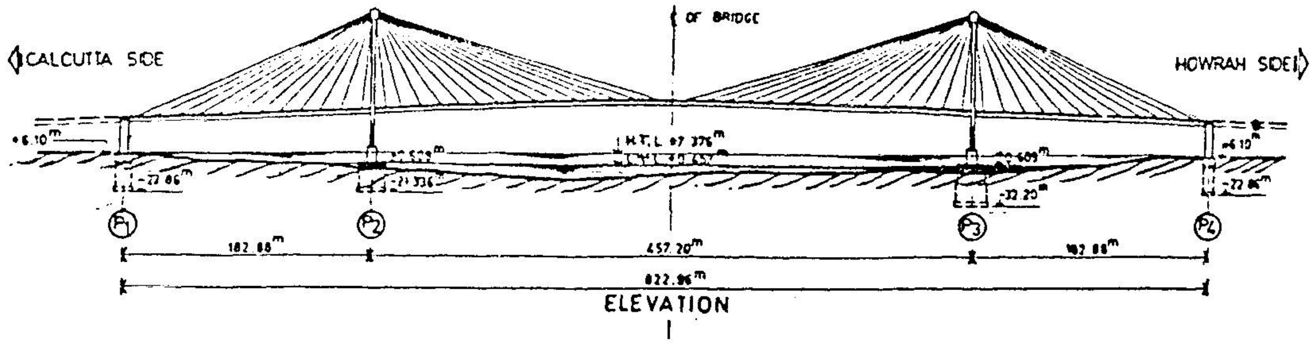
4.0 THE PROBLEMS OF F-1 & F-4 AND SOLUTIONS

Being land based foundations no extraordinary problem excepting during sinking the caissons, was encountered. The slight difficulty in sinking during the last few metres was overcome by providing a moderate kentledge. Normal underwater plugging was carried out on completion of sinking.

5.0 CONCLUSION

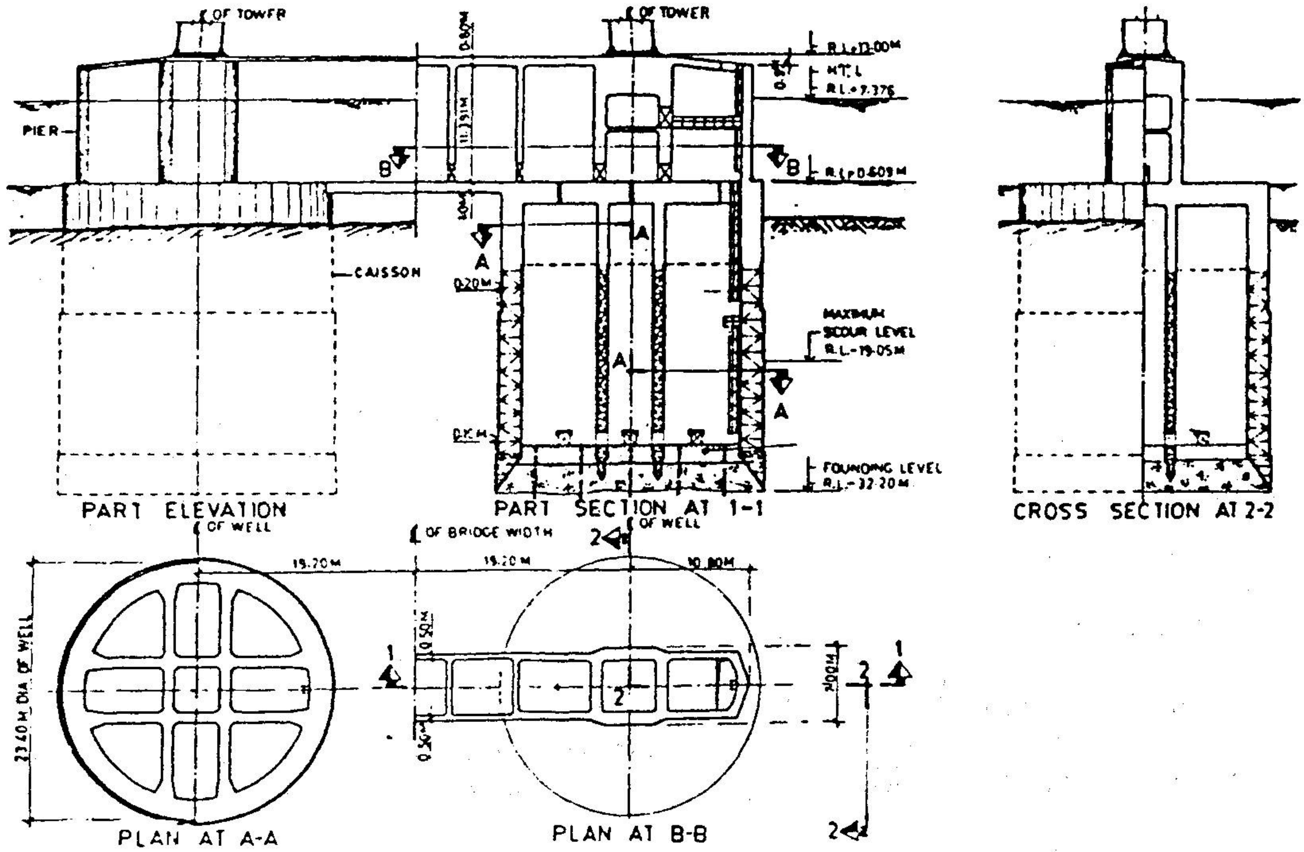
As described above, the construction of the foundations and substructure of 'Vidyasagar Setu' provided many challenges to the design and construction engineers. It can be said that the experience gained by the engineers in tackling the design and construction problems would provide the knowledge that could be drawn upon to help resolve similar problems expeditiously in future projects of this nature.

Acknowledgement: **Hooghly River Bridge Commissioners**



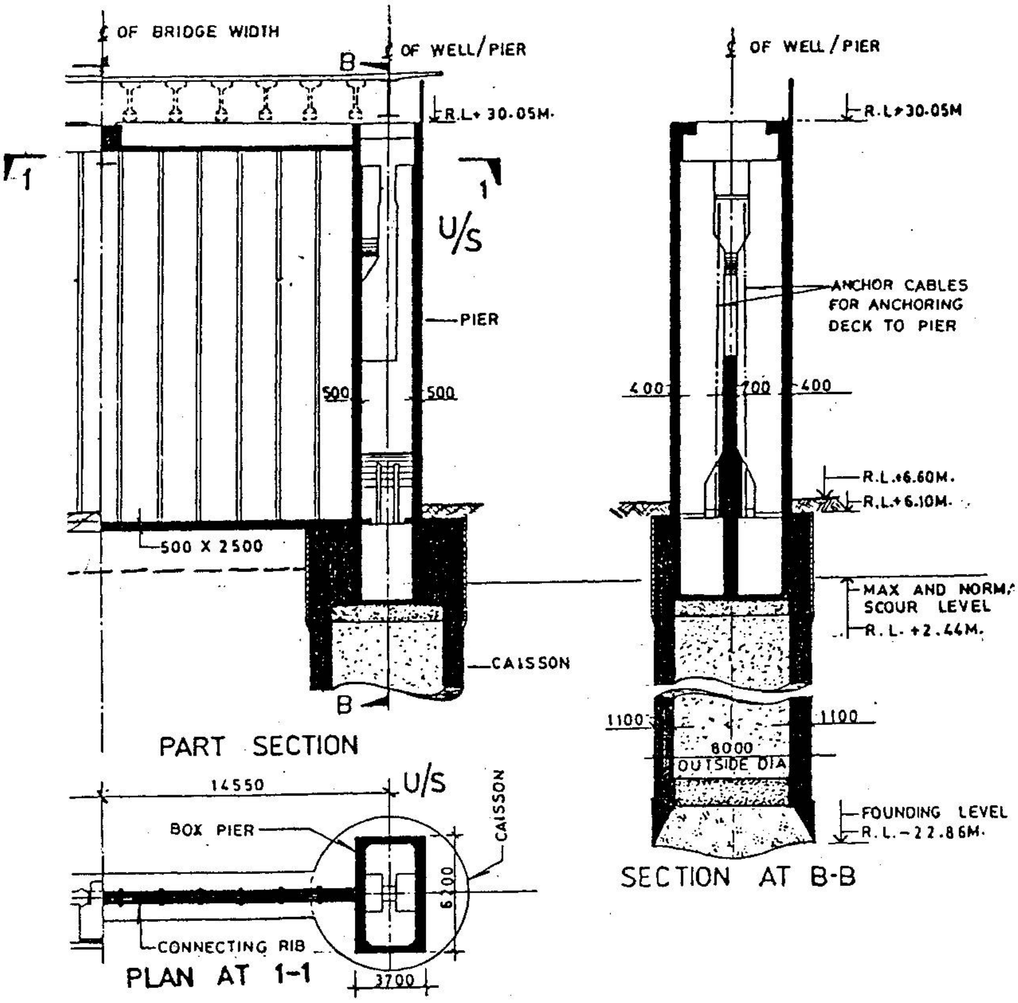
SECOND HOOGHLY BRIDGE

FIG. 1



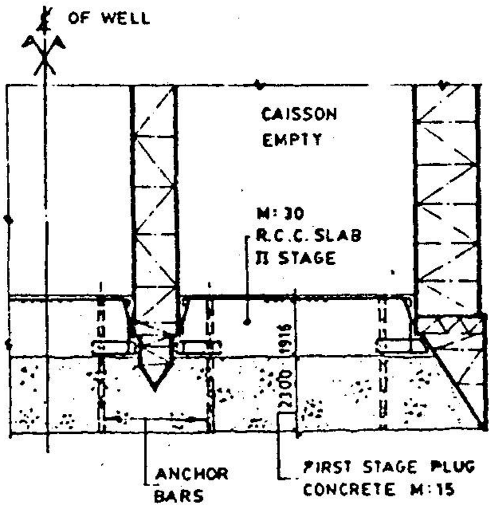
PIER P3 AND CAISSONS F3

FIG. 2



PART PIER P4 AND ONE CAISSON OF F-4

FIG.3



PART CROSS SECTION SHOWING BOTTOM SLAB REINFORCEMENT

BOTTOM PLUG FOR CAISSON OF F-3

FIG. 4

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DIFFICULT FOUNDATIONS OF JOGIGHOPA BRIDGE - SOME DESIGN ASPECTS

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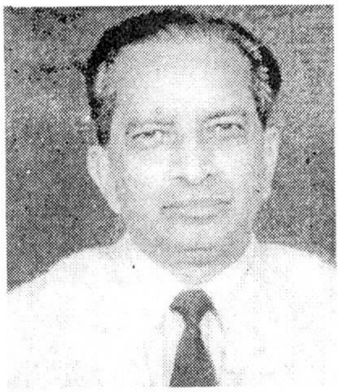

Born in 1941, he graduated in Civil Engineering in 1962 and joined Indian Railway Service of Engineers in 1963. He has been associated with survey and construction of new lines, doublings, gauge conversion, re-girdering/rebuilding of bridges etc. on various Zonal Railways. In his present assignment, he was closely involved in construction of Rail-cum-Road bridge at Jogighopa where Pier No. 17 & 18 had special foundation problems.

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Born in 1956, Shri Singh is a graduate in Civil Engineering and joined India Railway Service of Engineers in 1982. For the last 15 years he has been closely associated with construction of bridges over mighty Brahmaputra at Tezpur and Jogighopa under N. F. Railway Construction Organisation.

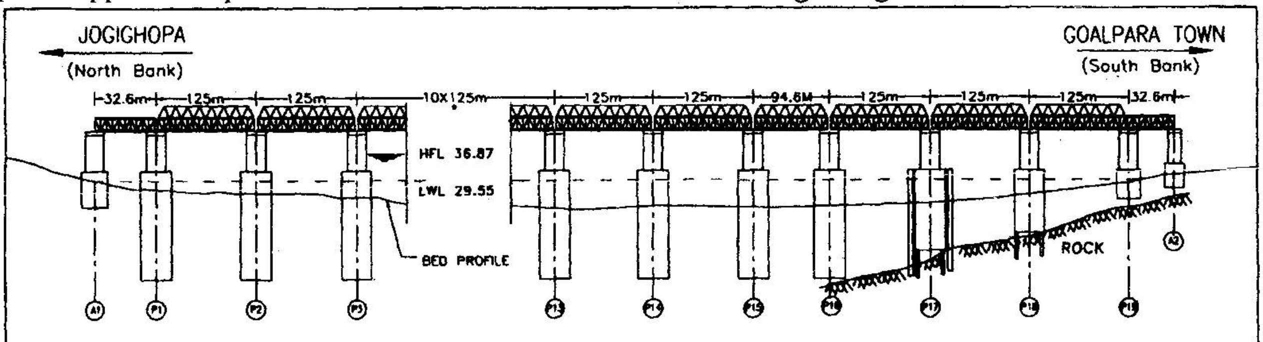
Abstract

The construction of the third bridge across the mighty river Brahmaputra was completed recently by the Indian Railways. The 2.30 Kms-long rail-cum-road bridge is situated at Jogighopa, 150 kilometres West of Guwahati, in the Bongaigoan district of the State of Assam. The work on the bridge commenced in December 1987 and it was opened to traffic on 15th April 1998. The bridge connects the existing Jogighopa Railway Station on the north bank to the Kamakhya Railway station near Guwahati by a 142 Kilometres long railway line traversing on the southern bank of Brahmaputra. The bridge also provides the road connection between the National Highway–31(B) on the north bank and the NH-37 originating from Pancharatna on the south bank of the river.

The site selected for the bridge was considered suitable in spite of problematic foundation conditions near south end of the bridge. The problem was caused by the presence of sloping rock at a depth of 45m below LWL, which is within the zone of anticipated maximum scour conditions. Two foundations, namely P-17 and P-18, had to be founded on undulating bedrock. Because of the difficult and typical foundation conditions encountered at these two locations, a special foundation system comprising of large diameter well and the anchor piles through the well steining was evolved. This paper briefly discusses the main design features for these two difficult foundations.

INTRODUCTION :

Jogighopa Bridge has a span configuration of 17 spans of 125 m each, one span of 94.60m and 2 shore spans of 32.60m each, all between the centres of the piers. The superstructure of the bridge consists of 18.50m high and 11.50m wide double Warren-type open-web steel girders. The roadway is on upper deck and the railway line is on the lower deck. All the foundations are well foundations. Abutments are supported on twin circular wells of 6.0m diameter having common well cap. 17 foundation wells are of double D type with plan dimensions of 11m x 17m. For separation of the road and rail traffic immediately beyond the main bridge, two road viaducts have been constructed. The north viaduct is 422m long and has 7 spans of 18m-RCC girders and 10 spans of 30m-PSC girders. The south viaduct is 660m long and has 7 spans of 18m-RCC girders and 18 spans of 30m-PSC girders. The substructure consists of hollow circular piers supported on pile foundations. The salient features of the bridge are given in the table below.



RAIL CUM ROAD BRIDGE AT JOGIGHOPA

Fig: 1

SALIENT FEATURES AT A GLANCE

Length of Main Bridge		:	2.284 Kms
Span configuration		:	1x32.6m+14x125m+1x94.60m+3x125m+1x32.60m.
Length of Road Viaducts		:	1.082 Kms
Span Configuration	(i) North Viaduct	:	7 x 17.86m (RCC) + 10 x 29.73m (PSC)
	(ii) South Viaduct	:	7 x 17.86m (RCC) + 18 x 29.73m (PSC)
Guide Bunds :	(i) North Bank	:	850 m long.
	(ii) South Bank	:	478 m long.

General Configuration :

Road on upper deck, Railway Line on lower deck.
Main spans – steel girders.
Double 'D' Type well foundations (11m x 17m).
Circular well foundations for P-17 and P-18 (18m dia).
Navigational Clearance 12.50 M above HFL.

Foundation Design Parameters :

Design discharge		:	90,400 cumecs.
Design discharge intensity		:	90 Cumecs/m max.
Max. velocity of flow		:	5.00 m/sec
Design HFL		:	RL (+) 36.87 m.
Seismic design HFL		:	RL (+) 35.87 m.
Low water level		:	RL (+) 29.55 m
Scour level :	Maximum	:	RL (-) 16.95 m
	Seismic	:	RL(-) 8.79 m
Design founding level		:	RL(-) 36.75 m
Buoyancy – well resting on sand		:	100%
	Well resting on rock	:	75%
	Maximum tilt	:	1 in 80
	Maximum shift	:	D/40

FOUNDATION PROBLEM :

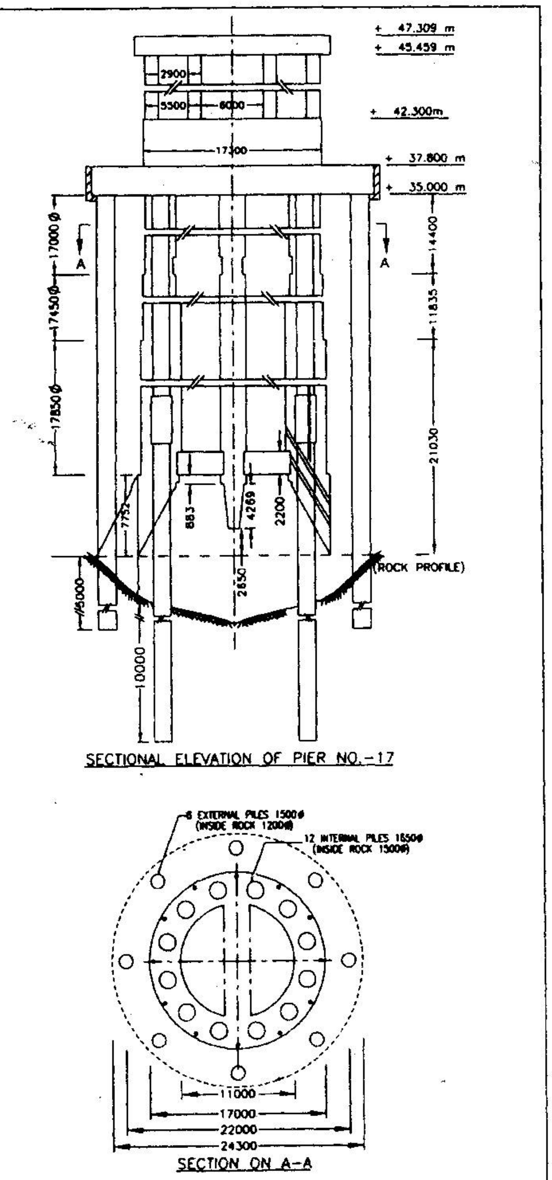


Fig:2

The normal well foundations have been sunk through the river bed material and are usually founded on the dense sand. The founding level is generally well below the deepest anticipated scour level so that the well has sufficient grip around the steining. During the worst scour conditions, the available grip around the well foundation prevents the overturning of the pier under the overturning moments caused by applied loads.

The detailed geotechnical investigations of the site conducted during the final location survey had revealed the presence of rocky strata at varying depths towards the southern side of the bridge alignment. Two foundations, namely P-17 and P-18 were affected by the presence of the undulating bed rock. The anticipated maximum scour was expected to reach upto the rock level leaving no grip around the wells. Due to the lack of lateral resistance, the foundations were unsafe under seismic conditions from stability considerations. Since the bedrock was 45 m below LWL, it was not possible to adopt pneumatic working for socketing the well into the bed rock. The design was governed by seismic forces. Since the earthquake can strike from any direction, a large size circular well was required. Since it was not possible to socket the well into the bed rock by further sinking, other means of positive anchorage of well with the bed rock was required.

The design of these foundations comprises of circular wells of 18m diameter with 12 numbers of 1.50m diameter anchor piles socketed through the well steining into the bed rock. The buoyancy effect at the rock base was taken @75% instead of usual 50%. The diameter of the wells has been reduced from 18m at the base to 17m at the well cap level. The anchoring of wells with the rock has been done with 12 nos. 1.50m diameter equally spaced reinforced concrete piles through the steining of the wells. The piles are anchored by 10m inside the rock and fully bonded with the well steining. The well has been provided with an 18m diameter solid M-30 grade concrete bottom plug at the base for transmitting the weight of the structure to the base rock. In addition to the internal piles, 8 numbers of 1.50m diameter external piles were also constructed at Pier No. 17 subsequently to strengthen it. These piles have been integrated with the well cap by a common well-pile cap. Various design aspects of the well base and internal and external piles are briefly discussed in the following sections.

Design Loads at Well Base

The dimensions of the well are shown in figure – 2. The total dead weight of various components of the foundations, substructure and the superstructure were worked out on the basis of section properties at different elevations. The weight of the superstructure including the road deck, railway track and the services is 2646t.

The experts at the University of Roorkee carried out the dynamic analysis of the structure for determining the seismic forces induced by the weight of the structure during earthquake vibrations. For the purpose of mathematical modeling, the entire pier structure was divided into 26 masses, which were lumped at appropriate levels. The pier structure has been modeled as a vertical cantilever structure free to vibrate laterally. The response spectrum for the site was developed on the basis of typical earthquakes having epicenters at different fault zones. Based on dynamic analysis, the longitudinal and transverse seismic moments and seismic shears at different levels were worked out. The vertical seismic coefficient was taken as g/16. The moments and shears at the base of the well under seismic conditions are tabulated below.

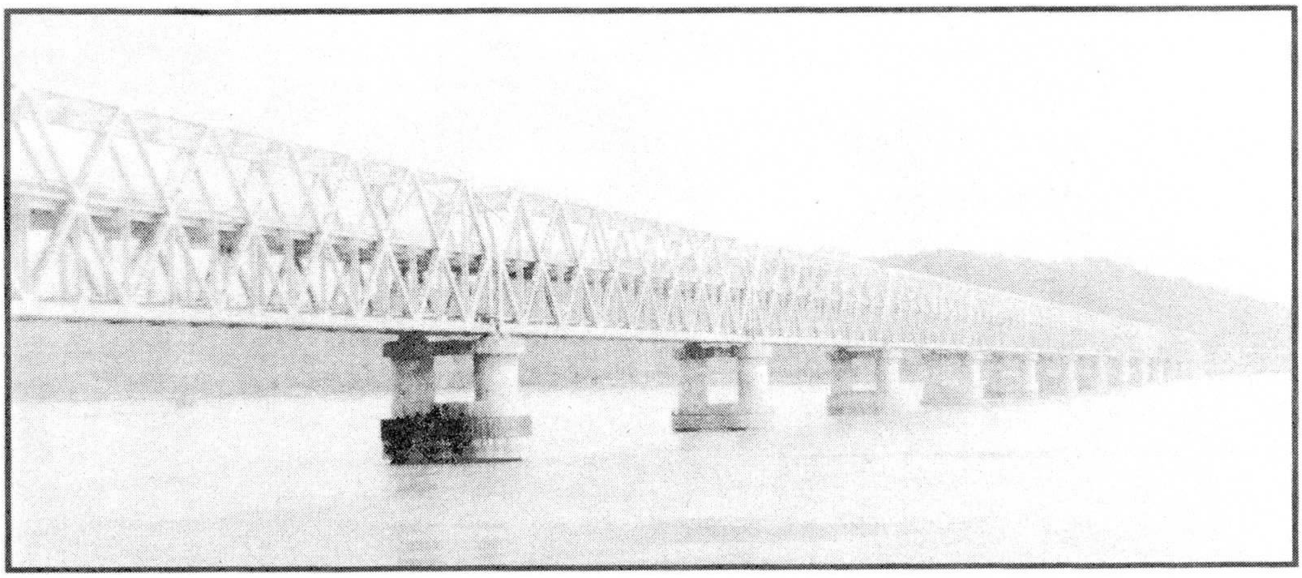


Fig:3

Design Loads at the Base of Pier No. 17

S N	Loads	Vertical Force (t)	Along the Bridge Axis: Force (t)	Along the Bridge Axis: LeverArm (m)	Along the Bridge Axis: Moment (t-m)	Across the Bridge Axis: Force (t)	Across the Bridge Axis: LeverArm (m)	Across the Bridge Axis: Moment (t-m)
1	Dead Loads	32355	Includes all dead loads of well, substructure and superstructure					
2	Buoyancy @75%	-9543						
3	Rly Live Loads	2034	-	-	-	-	-	-
4	Road LL @100%	366						
	@ 50%	183	Seismic case.					
5	Braking Force	-	335.3	63.192	21188	-	-	-
6	Water current	-	-	-	-	332	36.00	11952
7	Tilt & Shift	-	-	-	3742	-	-	-9710
	Total-Nonseismic	*25212*	*335.3*		*24930*	*332*		*2242*
8	Seismic-Horizontal	-	4519	-	187500	4130	-	174600
9	Seismic-Vertical	±2091	-	-	-	-	-	-
	Total-Seismic	**27120** **22938**	**4854**		**212430**	**4462**		**176842**

Analysis of the Well Base :

Normal Case : As already stated, the base of the well is 18m in diameter. The total section properties are :

Area = 254.47 m^2 Ixx = Iyy = 5153 m^4.

Maximum stress at the well base= P/A + (Mxx/Ixx). R + (Myy/Iyy).R

= (25212/254.47) ± (24930/5153) x 9.0 ± (2242/5153) x 9.0

= 99.08 ± 43.54 ± 3.92

= 146.54 t/m^2 = 14.65 kg/cm^2 - Maximum compressive stress

and 51.62 t/m^2 = 5.16 kg/cm^2 - Minimum compressive stress

Thus, no tension develops at base and the stresses are much below the permissible stress of 100 kg/cm^2 under bending compression for M-30 grade of concrete.

Seismic Case : In the seismic case, the base of the well is subjected to large moments (2,12, 442tm) in addition to the heavy vertical load of 22,938t. This combination of loads induces an eccentricity of about 9.26m. Because of the large eccentric load, the major portion of the base shall be subjected to tensile stresses. The analysis of the base as cracked RCC section is, therefore, called for. The results of the analysis of the well base under seismic condition are given in a tabulated form. The symbols used in the table are explained in the accompanying figure. The position of the Neutral Axis of the section was calculated by the process of successive iteration. To begin with, the location of the neutral axis has been assumed at the centre of the bottom plug. The sectional properties of the cracked section are then determined by standard formulae. With the load combination acting on the cracked section and the section properties thus determined, the revised location of the neutral axis is worked out and compared with the assumed location. The new location of the neutral axis is then taken as the basis for next iteration. After a few iterations, the correct location of the neutral axis is known along with the section properties of the cracked section. The stresses in the concrete and the steel are then worked out.

Axial Compression = 22938 t.

Applied Moment = $\sqrt{[(212430^2 + 2242^2)]}$ = 212442 tm.

Radius of bottom plug, R = 9.00m Modular Ratio, m_r = 16.20

Radius of internal piles, r_l = 0.75 m.

Distance of piles from centre of bottom plug R' = 7.25m and R_o = R'. Cos 15° = 7.003m.

Area of steel in each pile A_{stp} = 0.0550 m^2 (54 nos. of 36mm dia HYSD bars).

Gross area of each pile A_{cp} = 3.14/4*1.5^2=1.767 m^2.

Equivalent area of pile reinforcement = 16.2 x 0.055 = 0.891 m^2.

Determination of Neutral Axis (refer figure – 4)

The distance of the N. A. above the centre of the bottom plug (assumed), X_{na} = 2.82m.

Half the angle subtended by the Neutral Axis at the centre, $\alpha = Cos^{-1}(X_{na}/R)$ = 1.2512 rad.

Formulae for segment of circle :

Area = $(\alpha - Sin\alpha.Cos\alpha) R^2$; $Xcg = Xna + [(2/3).Sin^3\alpha/(\alpha - Sin\alpha.Cos\alpha) - Cos\alpha].R$

$Icg = (R^4/4).[\alpha - Sin\alpha.Cos\alpha + 2.Sin^3\alpha.Cos\alpha - \{(16/9).Sin^6\alpha/(\alpha - Sin\alpha.Cos\alpha)\}]$

Notations : I_{cg} = Moment of Inertia of the element about its own centroid; I_o = Moment of Inertia of the element about the centroid of the effective section; A_{eff} = Effective area of X-section after neglecting the concrete area in tension I_{eff} = Moment of Inertia of the effective section about the centroid of the effective section.

Sectional properties under load

Description	Area (m^2)	X_{cg} (m)	$A.X_{cg}$ (m^3)	I_{cg} (m^4)	$I_o = I_{cg}+A.(X_{cg}-X_o)^2$ (m^4)
1. Core concrete above N.A.	77.189	5.387	415.850	204.575	237.738
2. Pile No. 1 & 12 (steel)	1.782	7.003	12.479	0.000	9.190
3. Pile No. 2 & 11 (steel)	1.782	5.127	9.135	0.000	0.277
4. Pile No. 3 & 10 (steel)	1.782	1.876	3.344	0.000	14.531
5. Pile No. 4 & 9 (steel)	1.782	-1.876	-3.344	0.000	77.822
6. Pile N. 5 & 8 (steel)	1.782	-5.127	-9.135	0.000	173.193
7. Pile NO. 6 & 7 (steel)	1.782	-7.003	-12.479	0.000	245.398
$A_{eff} = \sum A$ =>	87.881		415.850		$I_{eff} = \sum I_o$ = 758.149

Distance of centroid of the effective section from centre, $X_o = \Sigma (A.X_{cg})/ \Sigma A = 4.732$ m.
Applied Moment about the centroid of the effective section, $M' = M - P.X_o = 103904.3$ t-m.
Distance of N.A. from centroid of the effective section, $Y_c = (P.I_{eff})/(M'.A_{eff}) = 1.904$ m.
Distance of Neutral Axis above the centre of circular section, $Y_o = X_o - Y_c = 2.828$ m.
(The distance of neutral axis, Y_o, as calculated above matches with the distance of the neutral axis, X_{na}, assumed above, since both are equal, the assumed position of neutral axis is correct.)

Calculation of Stresses in the Section

Max. comp. Stress in pile concrete = $\sigma_c = (P/A_{eff}) + (M'/Ieff).(R-Xo) = 845.93$ t/m²
Maximum tensile stress in pile steel, $\sigma_{st} = m_r\{M'/I_{eff}).(R_o+X_o)-(P/A_{eff})\} = 21825.8$ t/m²

Pile Forces

Max. comp. Stress in pile concrete $\sigma_c' = 845.93\ (7.003-2.828)/(9-2.828) = 572.22$ t/m²
Maximum compression in the pile = $\sigma_c'.\{A_{cp} + (16.2-1).A_{stp}\} = 1489.6$ t.
Maximum tension (uplift) in the outer most pile = $21825.8 \times 0.055 = 1200.4$ t.

The maximum tension in the piles is 1200t. Depending upon the earthquake direction, any pile can be subjected to the uplift of 1200t and hence all the piles have to be designed for the same capacity. The bottom plug and the piles are designed with M-30 grade concrete for which permissible stress under seismic condition is 1.333 x 100 = 133.3 kg/cm² under bending compression and 1.333 x 85 = 113.3 kg/cm² under direct compression. The maximum compressive stress in the bottom plug as well as pile concrete is well below these values and hence safe. The design of the anchor piles for the uplift capacity is given below :

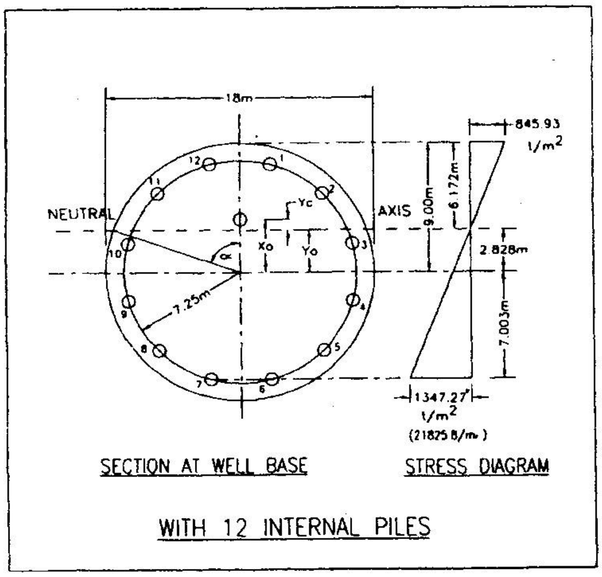


Fig:4

DESIGN OF ANCHOR PILES :

The piles have been provided essentially for resisting the tension developed at the base of the well caused due to overturning moments and resultant uplift under seismic conditions. The base of the well is treated as the solid circular section with each pile acting as steel reinforcement resisting the tension. The socket length inside the rock and the bond length of pile within the well steining have to provide adequate safety against the uplift. The brief computations are given below.

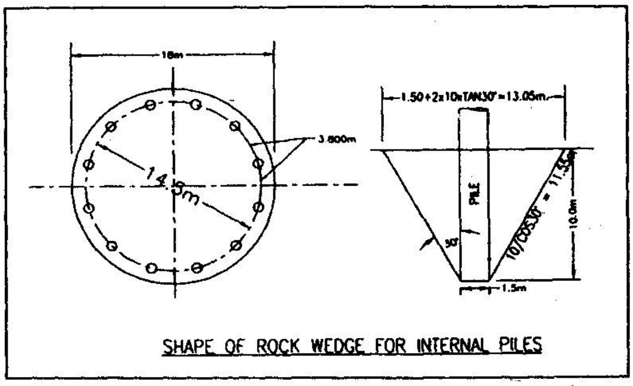


Fig:5

Pile Reinforcement

Maximum tension developed in the pile = 1200.4 t.
Use 36mm diameter HYSD bars. Area of each bar = 10.18 cm^2.
No. of bars per pile = 1200.4 x 1000/(1900 x 10.18 x 1.33) = 46.66 nos.
54 nos. of 36mm dia HYSD bars have been provided in each pile. This ensures the pile capacity of about 1392t in tension.

Pile Anchorage in the Rock (see figure – 5)

Max. uplift capacity of the pile = (1900 x 1.333) x 54 x 10.18/1000
= 1392.3 t (actual tension 1200.4t)
Assume piles in a row, the contributing length will be π x 14.5/12 = 3.80m
Anchorage length in the rock = 10m.
Width of the wedge at the top of the rock = 2 x 10 tan 30° + 1.5 = 13.05m
Buoyant unit weight of the rock @75% buoyancy = 2.30 – 0.75 = 1.55 t per cum.
Unconfined compressive strength of the rock = 253.4 kg/cm^2 = 2534 t/m^2.
Volume of the wedge = (1.50 + 13.05) x 10.00 x 3.80/2 = 276.45 cum
Buoyant weight of the rock wedge = 276.45 x (2.3 – 0.75) = 428.50 t
Shear stress = 0.01 x UCS = 0.01 x 2534 t/m^2 = 25.34 t/m^2.
Surface area = 3.80 x (10.0/Cos30°) x 2 = 3.80 x 11.55 x 2 = 87.88 m^2.
Shear resistance = 87.88 x 25.34 = 2224.3 t.
Vertical component of shear resistance = 2224.3 x Cos30° = 1926.3 t.
Total resistance = 428.5 + 1926.3 = 2352.1 t.
Load factor available = 2353.1/1392.3 = 1.90 > 1.5 OK

In the above calculations, submerged weight of soil above the rock-top upto the scour level has not been considered. This will provide additional safety against uplift.

Anchorage in the Well Steining

All the piles are fully bonded with well in the entire length of the steining. Out of 54 nos. of 36mm dia bars, 36 nos., have been curtailed after 10m of bond length in the steining. Beyond this only 18 nos. of 36 mm dia bars have been retained, which have been continued right upto the well cap.

EXTERNAL PILES AROUND P-17

This is the pier where the work of bottom plug and piling was done first. During the construction of the piles, it was found that the diameter of the bottom plug possibly did not cover the entire base of the well. This was attributable to improper cleaning of the dredge hole in the under water conditions. The jet grout columns surrounding the well base possibly did not seal the entire periphery of the well. This permitted the inflow of sand in the dredge hole under the high hydrostatic pressure of 45m during cleaning operations. The contact area of bottom plug with bedrock below was doubtful in the outer peripheral zone. After discussions with the experts, it was decided that the effective area of the truncated bottom plug of 14.0m diameter i.e. the area enclosed by the internal piles only should be considered in design. The integrity of the bottom plug in outer portion was doubtful and hence the foundations needed further strengthening. It was decided to construct 8 nos. of external piles around P-17 to strengthen this pier. 8 nos. of external piles of 1.50m diameter have been constructed outside the well in a symmetrical pattern and integrated with the foundation well by a common well cap. A socket length of 5m has been provided for these piles. The design of these external piles is covered under this section.

DESIGN OF EXTERNAL PILES:

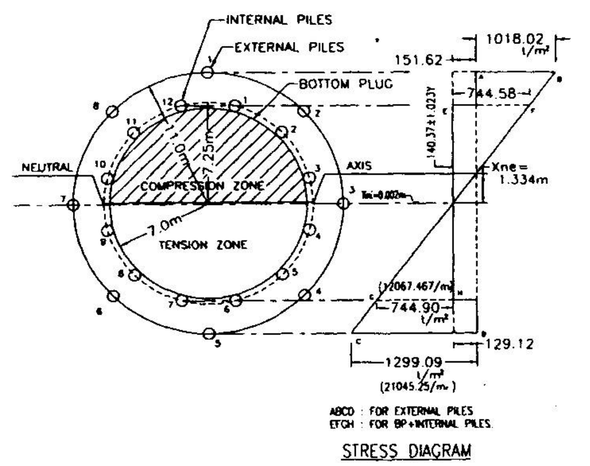
Basic of Analysis

For the design of the external piles, the fresh dynamic analysis of the pier has not been done. The pseudo-static seismic coefficients obtained from the earlier dynamic analysis were used for calculating the additional forces and moments induced at the well base due to the additional weight of the external piles and the extended well cap. The critical combination of the design loads is given below:

- Before construction of external piles i.e. all loads excluding the dead load of external piles and the extended portion of the well cap, all live loads and the seismic loads. P = 25,993 t. M = 2,773 tm.
- After the construction of external piles, i.e. full dead loads, live loads and seismic loads. P = 32,467 t. M = 2,42,057 tm.

The foundation well was found to be safe under the normal loading conditions without external piles. The external piles were required only under the seismic loading condition. The analysis of the combined well base has been done corresponding to two stages of loading. In the first stage of loading, the section consists of the well with the truncated bottom plug of 14m effective diameter and the full section of the 12 nos of internal piles. The loads operating at this stage are the full dead load of the well, piles, substructure and superstructure complete. At this stage full section is effective since the moments are small. It does not crack and therefore total section properties are applicable.

The second stage of loading will consist of the dead loads of external piles and the extended portion of the well cap, the live loads and seismic forces. At this stage, the external piles shall also be effective. The incremental effect of the second stage loads will be resisted by the composite section comprising of truncated bottom plug, internal piles and the external piles. Only the uncracked concrete along with the steel in the internal and external piles shall be effective in resisting the applied loads. Because of the composite action and the two stages of loading, the combined section will have two neutral axes, one for the initial section consisting of bottom plug and the internal piles and the other for the group of external piles. Both the stages of loading have been superimposed to obtain the final stress diagram. The position of the neutral axes has been found by trial and error. The compatibility of internal and external forces acting on the combined section of bottom plug and piles has been checked. The forces in the piles have been worked out from the final stress diagram.



COMBINED SECTION WITH 8 EXTERNAL PILES.

Fig:6

The pile forces in different internal and external piles are tabulated below :

Pile No.	Internal Piles	External Piles
1	1938.24	2650.05
2	1418.77	1766.72
3	519.04	-125.22
4	-177.95	-855.14
5	-485.91	-1157.49
6	-663.71	-855.14
7	-663.71	-125.22
8	-485.91	1766.72
9	-177.95	-
10	519.04	-
11	1418.77	-
12	1938.24	-

Design of External Piles

All the 8 external piles are of 1.50m diameter and are placed in a symmetrical pattern at 45° apart. These are reinforced by 54 nos. of 36mm diameter HYSD bars in a pattern similar to that of internal piles. The free standing length of the piles is 50m in the maximum anticipated scour condition. The effective length has been taken as 0.65L in accordance with Appendix-D of IS:456-1978.

The lateral shear acting at the bottom of the well cap has been assumed to be shared by the well and the group of external piles by frame action. The piles are fixed both at bottom and top. The well has been assumed as fixed at the bottom and free at the top. This is a conservative assumption for calculating shear in piles and gives higher shear force and consequently higher bending moment in the piles. The external piles have been designed for the tensile force of 1157 mt, compression of 2650 mt and the bending moment induced due to horizontal shear in piles. The socket length of 5.0m in the rock has been provided which is adequate for resisting the tension in the piles.

CONCLUSION :

Some of the important design features of the difficult well foundation P-17 and P-18 have been briefly discussed in this paper. These special design features were necessitated due to peculiar and difficult foundation conditions at site. The work has already been completed and the bridge has been opened to traffic.

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FOUNDATION FOR THE UDDEVALLA BRIDGE

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SUMMARY

The Uddevalla bridge, which is located approximately 80 km north of Gothenburg in Sweden, is an important part of the work to upgrade the E6 high way between Malmoe and Oslo to a motorway standard. The bridge has a total length of 1712 m and is composed of a cable-stayed bridge with two approach bridges. The main span has a length of 414 m.

The Uddevalla Bridge crosses both a major fjord and a small creek on the Swedish Westcoast. The geological conditions vary in a dramatic way. Soils formed during the last deglaciation extensively covers the solid rock. However rock surfaces are also occurring.

Parts of the bridge are founded in an end moraine with more or less firm silt, sand and gravel. The thickness of the till varies between 30 to 60 m. Foundations on footings as well as precast friction piles are used. The southern tower is supported by 106 steel pipe-piles with diameter 700 mm driven almost down to solid rock. The northern tower as well as some of the northern supports are founded directly on the rock surface. In the creek postglacial soft soils with organic clay and silt overlay glacial clay silt and sand deposits to a depth which varies up to 80 m. Steel pipe piles and steel core piles were used for the foundation of the supports placed in the creek.

The impact of the environment was a major consideration for the foundation work. Excavation in water is restricted in time and space with respect to the waste of sediment. The most determining requirement is however the maximum noise level in the surrounding area of buildings. To complete the pile driving it was necessary to make some extraordinary measures to reduce the noise level.

All foundation work was performed between August 1997 and March 1998 and the bridge will be opened for traffic in May 2000.

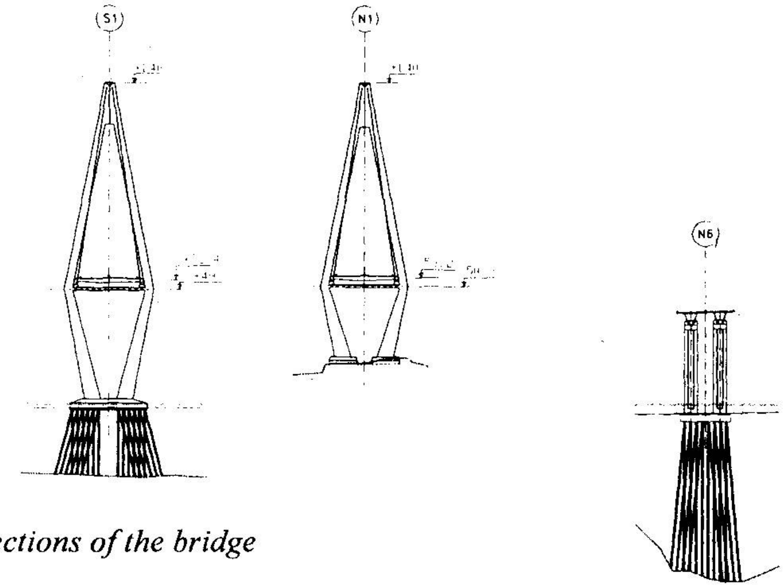

Figure 1b Sections of the bridge

Soil Conditions

The main span for the Uddevalla Bridge crosses the fjord "Byfjorden", while the northern approach bridge crosses the small creek "Kroken". The ground level above the water surface varies along the bridge from ±0 to + 37 m.

The soil layers along the southern approach bridge are parts of an end moraine. This formation has a length of about 2 km and a width of about 500 m. The supports S11, S10 and S9 are all placed in the central parts of the frontal moraine. To the north of these supports the bridge is located towards the western side of the centre of the end moraine. S8 to S5 and the southern tower S1 are thus founded in the western outer parts of the end moraine.

The northern part of the frontal moraine lies under water and forms the threshold of the fjord towards the sea. The frontal moraine was created during the last glaciation when temporary breaks occurred in the ice melting. The deposit was formed along the ice margin during a period of 100 to 200 years around 12 500 years ago. In a limited area moraine, silt, sand and gravel accumulated in one or several ridges in front of the ice. Fine-grained soils also sedimented when melt water from the ice flowed out at the ice front. Later when isostatic uplift occurred, the frontal moraine was exposed to waves and wave-wash, thus the soils vary in a dramatic way across the frontal moraine.

The thickness of the till, south of the fjord, varies between 30 to 60 m. Depth down to solid rock and its variation is indicated in figure 1. Sand is the dominating soil type down to 25 á 35 m depth. From S6 and north the sand is silty in the surface layer. The sand overlays silty clayey moraine.

The soil is dense to very dense at the location of S11, S10 and S9 according to results from hammer soundings. Further north the sediments are very loose to loose down to 10 á 15 m depth. Underneath these loose sediments the soil is dense to very dense.

North of the fjord where the northern tower is founded rock surface occurs. The rock type is mica rich gneiss.

The supports N5, N6 and N7 are placed and founded in the small creek. The water depth in the creek is limited to about 3 m. The depth down to solid rock is however up to 80 meters. Great variations in depth occur, as can be seen from figure 1, and locally the rock surfaces form very steep slopes. In this area the soil consists of postglacial sediments down to 20 m depth and

Introduction

The E6 highway between Oslo in Norway and Malmoe in the southern part of Sweden has during the last ten years been continuously upgraded to a motorway standard. The highway acts as an important link for the communication from Scandinavia to Europe.

One critical stretch is the by-pass of Uddevalla, which is located in the end of a deep fjord. The existing road passes east and through the central part of Uddevalla. After extensive investigations during the last thirty years it was decided that the new road will pass west of Uddevalla.

The new route is approximately 9 km and will save 12.8 km in total length for the road E6. However the route passes through very sensitive locations. The southern side of the fjord has both archaeological and geological values which are classified as of national interests. At the northern side of the fjord there is a location of established dwelling houses, which partly interfere with the suggested route.

The Swedish Road Administration performed a conceptual design and the tender work started in May 1996. The accepted tenderer was the Swedish contractor SKANSKA and the Design and Build contract was signed in January 1997. Structural and geotechnical design was performed by Skanska Teknik. The design work started immediately and the construction works started in August 1997. The bridge will be opened for traffic in May 2000.

The Bridge

The bridge has a total length of 1712 m and comprises of three different parts. The southern part is an approach bridge with a length of 506 m. The central part is the main bridge with a total length of 772 m and finally the northern part is an approach bridge with a length of 434 m. The bridge width is designed for a four lane motorway, i.e. a free width of 11.25 m in both directions.

The two approach bridges are composed of two parallel steel box girders with an upper composite slab of concrete. Each girder is supported on six piers and one abutment. The two piers for a support are normally founded on one common footing. The foundation is performed with footings on soil, solid rock or piles.

The main bridge is a cable stayed bridge having a main span of 414 m and two side spans of 179 m each. The superstructure is a steel grid with a composite slab of precast concrete. The two towers have a height of 140 m above the sea level. The towers are built in concrete using self climbing formwork. The southern tower is founded on piles and the northern tower is founded on solid rock.

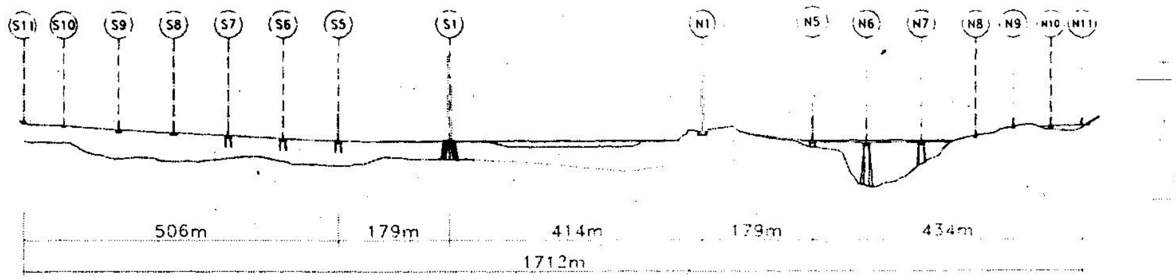

Figure 1a Elevation of the bridge

underneath follows glacial sediments. The postglacial soils are in the surface layers sand with gyttja, clay and silt. The dominating postglacial soil is however loose clay. This clay is described in figure 2. The glacial sediments are sand and clay with sand layers.

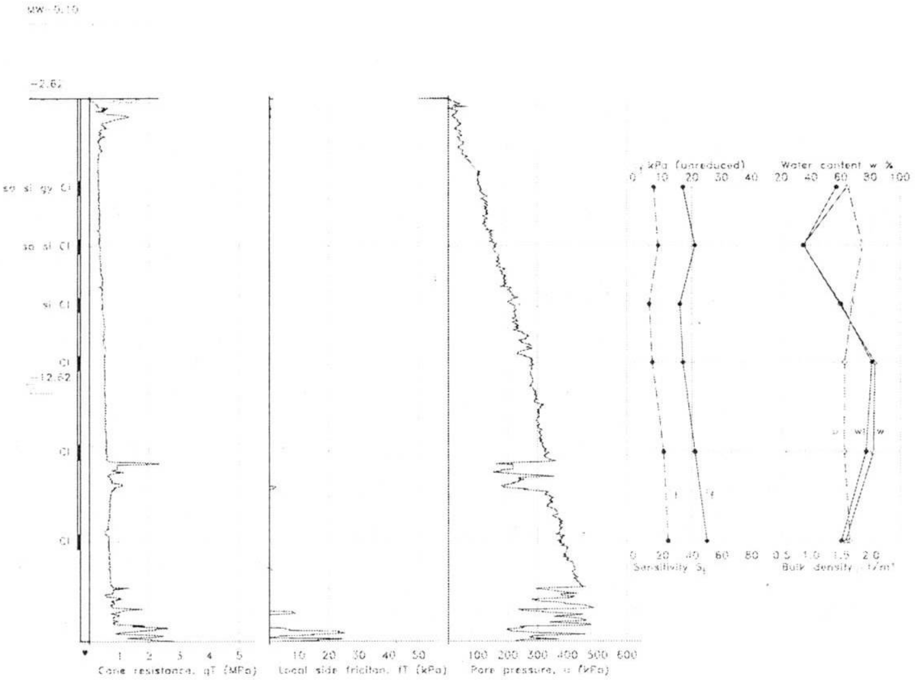

Figure 2 CPT-results and results from laboratory investigations at support N6

North of the creek the rock surface is dominating along the bridge line. However, in the vicinity of support N10 thin soil layers occur locally. The rock type is mica rich gneiss near the creek and north support N8 the rock type is massive granite.

Foundation

The supports S11, S10, S9 and S8 are founded directly on soil with footings with an area of about 10x20 m^2 each. The original loose soil under the slabs for S10 and S8 was exchanged for compacted gravel down to 3 m respectively 1 m depths. S11 and S9 were however founded directly on the original soil. Maximum vertical settlement calculated in serviceability limit state is 50 mm for these four supports founded with footings .

The supports S7, S6 and S5 were founded on driven friction piles. Precast concrete piles with square cross section 350x350 mm were used. Approximately 70 piles were driven for each support. Pile capacity was determined with dynamic testing including stress wave measurements. A Pile Driving Analyzer was used together with strain gauges and accelerometers attached to the pile head. CAPWAP analysis of the measured signal was performed to determine the static pile capacity and its distribution along the shaft and on the toe. Approximately 10 % of the piles were tested. Pile lengths varied between 8 to 26 meters.

The southern tower, S1, is supported by 106 steel pipe piles having a diameter 711 mm. The wall thickness of the pipe is 14,2 mm. The pipes were driven almost down to solid rock and then filled with concrete. Foundation for S1 with steel pipe piles was originally proposed by the client in the conceptional design. Bored piles were suggested by the contractor in the tender as an option. However, the client favoured steel piles. Pile capacity was determined by dynamic testing in accordance with the same procedure as for the concrete piles. The required pile capacity in ultimate limit state is 4500 kN. Calculated vertical deformation in serviceability limit state is 7 mm. The very limited deformation is due to the fact that most of the pile capacity origins from the toe located near solid rock.

The northern tower, N1, is founded directly on the solid rock. Maximum allowed rock pressure in ultimate limit state is 3,9 MPa.

The support N5 is founded with 41 steel core piles drilled down into solid rock. The steel core diameter is 150 mm and 210 mm for some of the piles and the pile capacity is 2500 respectively 3600 kN in compression. In tension the required pile capacity is 800 kN for both types of piles. The steel core is surrounded by a steel-pipe filled with concrete.

The two supports N6 and N7 in the small creek are both founded on driven steel pipe piles filled with concrete. About 50 piles were used for each support. The pile diameter is 508 mm and the wall thickness is 14,2 mm. The pile capacity is 2250 kN. The piles are driven down to the rock surface and thus end bearing. Due to the steep rock surface the toes for 15 % of the piles are secured with a dowel drilled into the rock. Dynamic testing was used for determining the pile capacity.

The supports N8, N9 and N11 are founded directly on the rock surface.

The support N10 rests on a footing founded on compacted fill lying directly on the rock surface. The thickness of the fill is up to 7 meters. The compacted fill consists of friction material.

Performance of the foundation work

The available time for construction is very short. Therefore it has been necessary to start the foundation works for several supports simultaneously. These works started in August 1997 and were completed in March 1998.

As mentioned in the introduction the bridge is placed in a very sensitive environment in many respects. Foundation works in general have a significant impact on the surrounding area. Noise, pollution and destruction of archaeological and geological values are common negative effects. To avoid these effects the contractor has had to present and follow, for Swedish regulations, an extensive environmental plan. The performance of the foundation work has been strongly influenced by this plan. The requirement for maximum noise level is based on an equivalent noise level, L_{Aq}, in dB(A). The requirements for different times during a week are shown in the table below. $L_A(t)$ is the noise level in dB(A) at time t.

	Mon-Fri 07-18	18-22 & Weekends	22-07
Dwelling houses	$L_{Aq} \leq 60$	$L_{Aq} \leq 50$	$L_{Aq} \leq 45$

$$L_{Aq} = 10 \cdot \log \frac{1}{T} \cdot \int_0^T 10^{\frac{L_A(t)}{10}} \cdot dt \text{ [dB(A)]}$$

The southern approach bridge is founded with footings and piled foundations. The piling work started in a support close to the core of the end moraine (S8). The support was originally designed with friction piles. However the piling work was soon interrupted because it was impossible to continue the work without destroying the concrete piles during the installation.
After additional geotechnical investigations it was decided to change the foundation to a footing founded directly on the soil. The remaining supports are performed in principal according to the initial design.

The two towers for the main bridge are founded with footings on steel pipe piles and solid rock. The installation of the 106 steel pipe for the southern tower were assumed to imply an extensive pile driving. The work was planned to be performed with an accelerated hammer with a mass of 7 tons and a theoretical energy of 75 kNm. The work started in August 1997. After a couple of weeks it was obvious that the pile driving equipment wasn't efficient enough with respect to the time schedule. It was common with pile-stop after 20 m quite hard driving without reaching the rock. After some days without driving it was possible to drive the pile further. So called "false refusal" was a matter of fact.

To increase the capacity of the pile driving it was necessary to take alternative measures to decrease the amount of blows for the installation with the originally selected equipment or change the equipment to a more efficient one. However the allowed noise level was reached (and in some cases exceeded) with the initially selected pile driving equipment. To avoid exceeding the stated noise level the first alternative was to reduce the amount of blows during the installation through drilling beside the pile during the driving. High water pressures had been observed during the pile driving. The drilling reduced the pore pressure and had a good effect on the capacity of the pile driving, but not good enough. Hence it was decided to add an equipment with a more efficient hammer to make the final driving of the piles. That implied that it was also necessary to take additional measures to increase the noise damping. These were performed in two ways. The first was a tube of concrete with a length of 5 m placed around the pile and the hammer during the final pile driving. The second way was a wall of normal transport containers with a height of about 10 m placed on three sides of the foundation area. Together these measures were efficient to reduce the noise level so it was possible to perform the piled foundation without exceeding the stated maximum noise level in any apparent way.

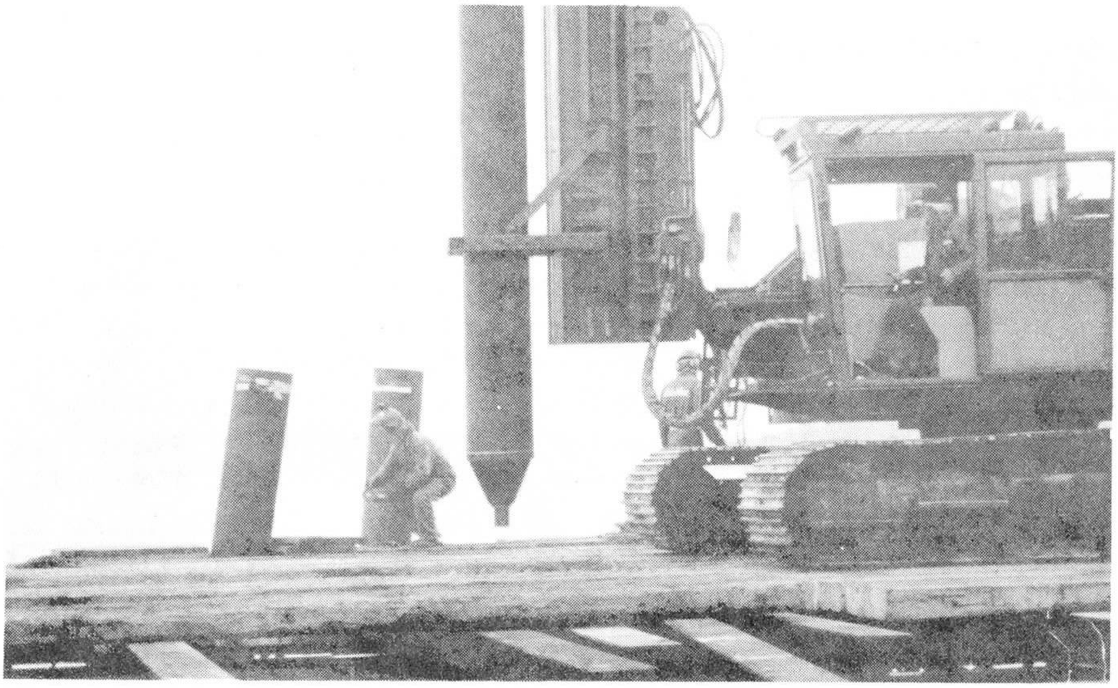

Figure 3. Pile driving equipment used for foundation at support N6 and N7.

The northern approach bridge is founded with footings on solid rock, in soil and on piles driven to rock. The additional geotechnical investigations for the piled foundations (support N5, N6 and N7) indicated that the rock surface locally has steeper slopes than originally assumed. Hence some of the pile toes were redesigned with a steel dowel as mentioned above. The foundation works were performed in principal in accordance to the original design. Compare photo in figure 3 showing the equipment for pile driving of the steel pipe piles at support N6 and N7.

Planning and Monitoring the Foundations for the Øresund Bridge

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SUMMARY

The Øresund Bridge across the sound between Denmark and Sweden is under construction. The road and rail bridge is 7.8km long and consists of a central 1.1km cable-stayed bridge with a main span of 490m. The approach spans are typically 140m. The pylons and many of the 51 piers have to withstand large ship impact forces and all are designed for winter ice loading.

The construction contract for the bridge is a modified 'design and construction' contract. The information supplied at tender stage by the Owner, Øresundskonsortiet and his consultants, contained, in addition to the usual Design and Construction Requirements, also a set of geotechnical Reference Conditions, a set of Definition Drawings and an Illustrative Design.

The geotechnical Reference Conditions contained the ground stratigraphy and a summary of the strength and deformation properties for the ground. The Definition Drawings fixed all visible dimensions for the bridge - foundation types and sizes were not shown. The Illustrative Design was included for information and showed in details a solution fulfilling all the Owner's requirements – approximate foundation details were shown.

The geotechnical Reference Conditions form the basis for all geotechnical design work. Bands of uncertainty were provided within the Reference Conditions and the Contractor accepted these bands as "foreseen" ground conditions. Where the ground conditions were outside the Reference Conditions intervals, the Owner accepted the risk. In only five of the 53 foundation locations were the ground conditions outside the predefined Reference Condition intervals; all of these instances referred to strata levels rather than actual soil or rock strength and deformation properties.

ASO Group is responsible for the bridge design and is the Owner's bridge consultant. During the construction phase ASO is monitoring and auditing the Contractor's work to ensure that the Owner's requirements on quality are fulfilled. ASO Group consists of Ove Arup & Partners (GB), SETEC (F) and Gimsing & Madsen and ISC (DK).

1 INTRODUCTION

The Øresund Link across the sound between Denmark and Sweden is under construction. The Link is owned by Øresundskonsortiet who, with advise from in-house consultants, have let four design and build contracts for the main section of the Link to contractor led consortia. The contractors are responsible for design as well as the actual construction of the Link. The Link is due to open during the year 2000.

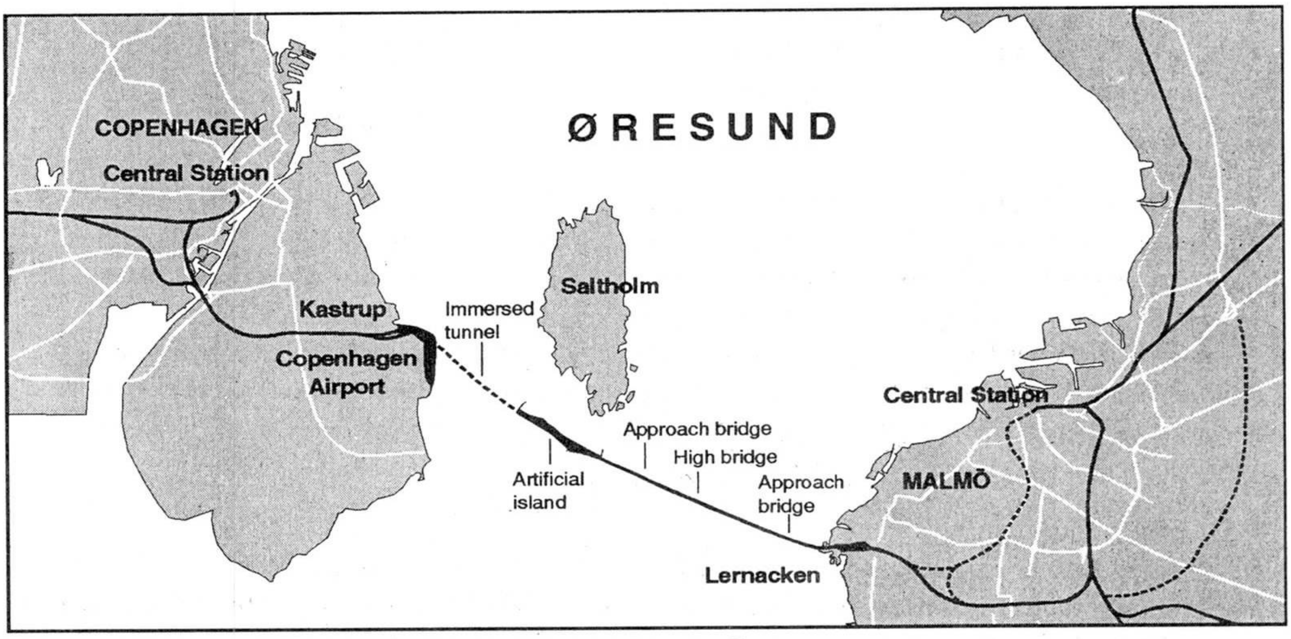


Fig.1. Plan of the Øresund Link

The 15.8km Link comprises, from Copenhagen Airport in Denmark to Lernacken in Sweden, the following main parts (Fig.1):

- a 0.4km reclaimed peninsula,
- a 3.5km immersed concrete tunnel;
- a 4.1km artificial island, and
- a 7.8km bridge, consisting of two approach bridges and a cable-stayed high bridge.

The bridge deck carries the traffic at two levels with the dual two-lane motorway at the upper level and the two tracks for the high-speed railway at the lower level. The bridge superstructure is supported on 51 piers, two pylons 203.5m high and two abutments, one on the artificial island and the other at Lernacken.

This paper describes the Owner's contract strategy in particular with regards to the definition of ground conditions in the form of geotechnical Reference Conditions.

The construction contract for the Øresund Bridge was signed in November 1995 between the Owner, Øresundskonsortiet and Sundlink Contractors consisting of Skanska (S), Hochtief (D) and Monberg & Thorsen and Højgaard & Schultz (DK). The Contractor's detailed designer is CV JV consisting of COWI (DK) and VBB (S).

2 CONTRACT STRATEGY AND DOCUMENTS

2.1 Contract Strategy

The construction contract for the Øresund Bridge was a modified design and construction contract. Prior to the tender stage the Owner and his consultants carried out extensive investigations into the physical conditions along the bridge alignment. These investigations allowed characterisation of the ground conditions prevailing at the site.

2.2 Contract Documents

The Contract Documents provided adequate information on which all tendering consortia could provide a fixed price tender bid. The main documents concerning the design and construction of the bridge foundations were:

2.2.1 Reference Conditions

The Reference Conditions contained a description of the weather and ground conditions (mainly clay and sand tills over layers of limestone), that could be encountered at the site. This ground condition information included a summary of the stratigraphy (the variation in surface level of each stratum along the bridge centreline) and the strength and deformation properties of the stratum (mainly till and limestone); the information provided was sufficient to design the bridge foundations. The Reference Conditions also presented index property test results that demonstrated the variability of materials encountered (especially in the limestone strata). These properties allowed characterisation of the material (based on relatively inexpensive laboratory and descriptive techniques) during the Contractor's ground investigation. The information was provided at tender stage and was used during the basic design stage of the bridge foundations; at this stage of the design process pier specific ground investigation was not yet available. The soil and rock parameters contained in the Reference Conditions were also the basis for the detailed bridge foundation design. However, at the detailed design stage, the Contractor was required to demonstrate (by means of his site specific ground investigation drillings) that the information provided in the Reference Conditions was valid for each and every pier location. Detailed information on the Owners pre-tender ground investigation work carried out during the preparation of the Reference Conditions document are contained in Volume 5 of the 11th European Conference on Soil Mechanics and Foundation Engineering including [1] and [2].

The weather conditions were defined in terms of current, wind and temperature all three of which influence construction activities offshore.

2.2.2 Design Requirements

The contractual design requirements consisted of three parts: Design Requirements, the Project Application Document and the Definition Drawings. As a basis for the design the Eurocode system was adopted. However, not all the relevant Eurocodes were complete at the time of tender, and many existed only in draft form, so to adapt them to this project a set of Project Application Documents was prepared giving amendments to individual Eurocodes (e.g. EC7 for foundations). Partial safety factors were calibrated, and accidental load cases identified and defined to satisfy the operational risk acceptance criteria developed for the completed Link. The Design Requirements document listed specific load cases and methods for calculation of these loads. Critical load cases for foundation design were ship impacts for the central piers and pylons and the serviceability limit case for all piers/pylons.

The Definition Drawings defined all visual geometry of the bridge; there was little geotechnical input here. A parallel document to the Definition Drawings was the Illustrative Design, which was not part of the contractual documentation but included for information only. This set of drawings showed an example design for the bridge conforming to the design requirements.

In terms of review of the Contractor's design the Owner retained the right to approve the basic design for conformance to the Design Requirements. However, the Owner only retained the right for comment on the detailed design and thus did not perform the role of checker. This firmly laid the design responsibility on the Contractor.

2.2.3 Construction Requirements

This document contained the requirements (the minimum requirements) for all anticipated construction activities. Regarding the foundations there where structural related requirements for concrete, there were also requirements for the detailed ground investigations at each pier/pylon location and requirements for the underbase grouting and full-scale trials of this grouting between the underside of caissons and the excavated limestone surface.

The requirements for the detailed ground investigations were such that, assuming good execution of the drilling work, the index properties of the sampled ground (density, point load index and unconfined compressive strength etc.) could be compared with the parameters that were measured during the formulation of the Reference Conditions. This approach reduced the necessary ground investigation at each pier location to a minimum by reliance on the vast amount of pre-tender detailed information that was correlated with the design parameters.

The necessary bond between the bases of the caissons and the intact Limestone was to an extent a function of the Contractor's design. However, even with this condition it was considered necessary to state that a uniform contact between the caisson base and the intact Limestone must be achieved. To enable the Contractor's construction method to be demonstrated the Contractor was required to carry out a full-scale trial (in fact large scale) of the grouting procedure. To enable the robustness of the trial to be proven the trial was required to be carried out successfully twice. The trial grouting area was 8m by 11m in plan and represented the bases of both the pylon caissons (rough) and the pier caissons (toothed). It is noted here that the requirement for trial grouting was so strict that when, on one of the pylon caissons, imperfections were discovered beneath the caisson the need to execute trials of the remedial measures was an accepted part of the repair process.

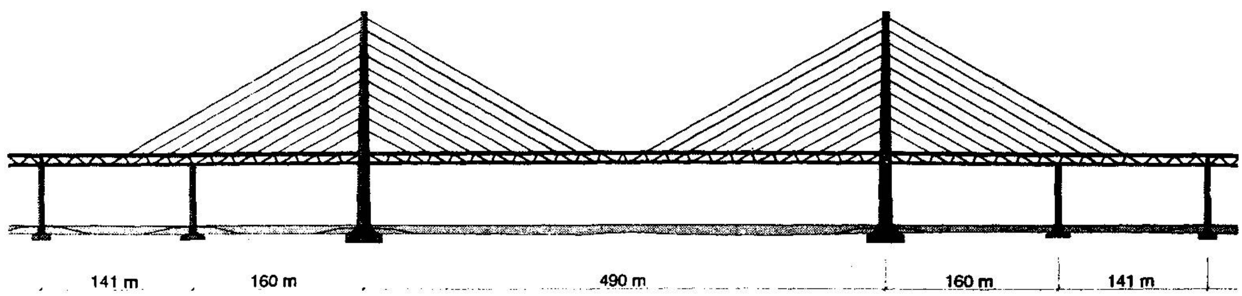
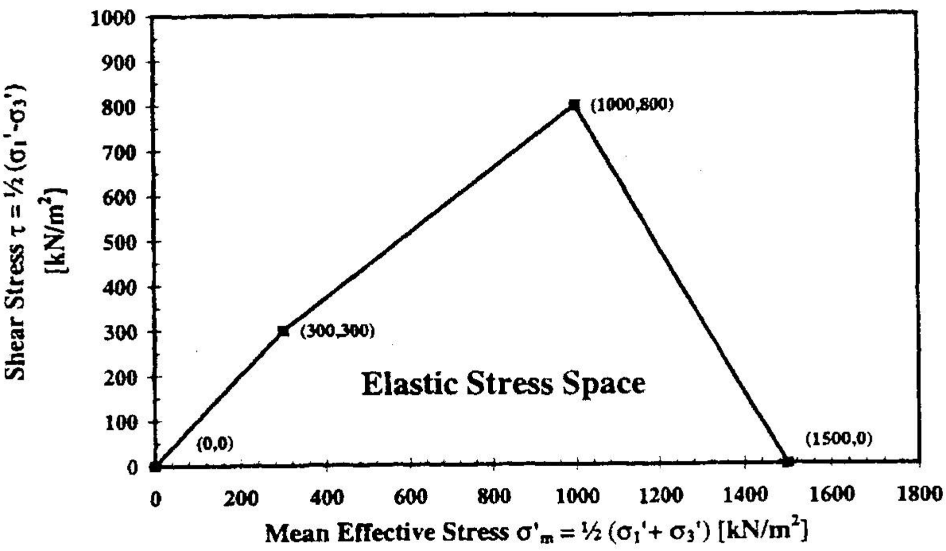


Fig.2 The 490m Main Span crosses an international Shipping Lane

3 MAIN FOUNDATION DESIGN CONSIDERATIONS

The depth of water along the bridge alignment varies from approximately 2m to over 8m. At the high bridge pylons the water depth associated with the main shipping water level resulted in ship impact design forces of up to 560MN being appropriate. Although the pylons were surrounded by protective islands (Fig.2) they were still required to be able to resist the full ship impact force of 560MN ignoring the effect of the protective island. The three adjoining piers were also protected but not required to resist the impact irrespective of the islands. Unprotected piers were designed for ship impact forces up to 211MN. These large horizontal forces combined with the anisotropic strength of the limestone foundation (weak and weathered horizontal layers) resulted in an accidental impact loading that was in some instances the critical load case. Design of the ship impact resistance needed to limit the permanent bridge displacement to 100mm requiring dynamic calculations used whereby the caisson's inertia, movement (acceleration etc.) and the shear resistance of the ground was related to the impact energy and stiffness.



Where :

σ_1' : **Major Effective Principal Stress**

σ_3' : **Minor Effective Principal Stress**

Fig.3 Definition of Elastic Stress Space for Limestone Formation

For all caissons regardless of seabed level a critical load case was that of cyclic degradation of the Limestone foundation material appropriate to the serviceability limit state. The specified design check for this required that the stresses mobilised in the limestone were within a predefined elastic yield loci with bounds related to both shear stresses and normal stresses (Fig.3). The Design Requirements stated that all stress states within the yield loci would result only in elastic strains and thus limit any cyclic degradation of the limestone to a minimum.

For ship impact loading, the anisotropic nature of the limestone was of great importance. Hence, the nature of the site investigations mentioned above was such that low strength layers (called H1 layers of completely weathered limestone) of 100mm or greater had to be identified by means of core recovery and downhole geophysical testing during the Contractors site specific ground investigations. Similarly, occurrence of weak layers immediately beneath the foundation level would invalidate the Elastic Stress Space and would result in large and possibly uneven settlement of the bridge foundations.

In both of the critical load cases the information supplied to the contractor led to the design criteria being easily understood without the need for a comprehensive investigation into the behaviour of the limestone being undertaken during the detailed design stage of the contract.

4 ACTUAL GROUND CONDITIONS AND CONSTRUCTION

4.1 Revealed Ground Conditions

The geological boundaries (the important ones being the seabed level and level of the top of limestone) specified in the Reference Conditions document were given to an accuracy of ±1.5m. Any variation of the level of less that 1.5m from the Reference Conditions was deemed to be at the Contractor's risk whereas any variation beyond 1.5m was deemed to be at the Owner's risk.

As part of the Contractor's design he was allowed to alter the pier locations within certain geometrical constraints. Hence, the pre-tender borings commissioned by the Owner did not, nor were intended to, survey all pier locations. Interpolation between borehole positions was aided by geophysical survey techniques.

In order that there was a thorough check of the information contained in the Reference Conditions the Contractor was required to carry out a specific site investigation at each pier location. The nature of the site investigation was specified in the Construction Requirements document in terms of both borehole dimensions, depth, type of drilling equipment and the regimes for sampling and testing of samples. The specified methods of investigation had previously been tested during the Owners pre-tender investigations and were seen to be the most appropriate for the anticipated ground conditions. The Contractor's ground investigation was not intended to redefine the Reference Condition ground parameters, but was aimed at justifying their use during the detailed design of the bridge. The investigations carried out during the formulation of the Reference Conditions involved sophisticated large scale laboratory testing and complimentary quarry plate load tests, neither of which were part of the Contractor's investigation.

At only 5 of the 53 offshore foundation positions were the Reference Conditions seen to be inaccurate with respect to strata levels. The most notable, though not largest, variation in level was at the West Pylon where the surface level of the Limestone was seen 1.5m lower than the Reference Condition median level. The result of this was that the approved basic design needed to be modified to account for this strata level change. The cost of the physical change was absorbed by the Contractor, the cost of the redesign was, however, absorbed by the Owner due to bad weather conditions delaying the availability of site investigation information. This meant that the Contractor had to start the detailed design without having the results of his site investigation. At the five most easterly caisson locations the Limestone was observed by the Contractor to be more porous and to have thicker highly weathered bands than previously anticipated i.e. that the Reference Conditions were invalid. It was not suggested that the caisson foundation would fail but that larger than usual settlement (and differential settlements) could occur. The Owner noted the concern of the

Contractor's designer and, without agreeing that abnormal ground conditions were present, accepted that precautionary measures should be adopted to limit the implications of larger than the assumed settlements of 30 to 50mm. The design solution was to construct the bearing plate oversized making allowance for possible future shimming and if necessary realignment of the bridge bearings in case larger movements occurred when the full dead load was transferred. The cost of this extra bearing size was shared between the Contractor and the Owner. The Owner accepted that should further modifications be needed he would himself absorb these extra costs. In the end no additional measures were required and the caisson behaviour of the five caissons in question was seen to be similar to other caissons.

4.2 Construction Quality

The Contractor is responsible for the quality of the permanent works. To enable the quality to be monitored by the Owner the Contractor was required to seek approval for his Quality System and his detailed Quality Plans covering all design and construction activities. On site monitoring and assessment of the Contractor's performance by both the Contractor's own quality management and the Owner's Consultant ASO Group, is then carried out on the basis of these approved Quality Plans, the Construction Requirements and the Working Drawings. That the Contractor is responsible for the quality of his own work means that if he finds that the quality is not as required or procedures are not adhered to, then he is required to issue a non-conformance report (NCR) identifying the non-conformance and the proposed corrective action. In case of remedial work being required, approval is required by both the Contractor's designer and the Owner prior to the remedial work being carried out. In the case of the West Pylon foundations faulty underbase grouting was found. The Contractor had located the bad grout, carried out tests and subsequently filed an NCR. The corrective action, which consisted of revisions to method and procedures for the future grouting operation, was agreed as was the remedial measurements required to the actual substandard grout. In other instances where the Owner's Consultant identifies an apparent non-conformance (ANC), the fact is drawn to the attention of the Contractor, and when agreement is reached as to the status of the ANC the Contractor issues an NCR similarly to him having identified the non-conformance himself. Hence, in all instances the Contractor remains solely responsible for the quality and for satisfactory remediation of non-conformances.

5 CONCLUSIONS

The strategy adopted by the Owner with respect to the way in which the ground conditions were treated for the Øresund Link project can be summarised as follows:

- Carry out detailed pre-tender site investigations and interpret all data to be included in the Reference Conditions;
- Accept the risk, and cost/time implications, should the ground parameters and stratigraphy levels found at each pier location be outside the limits of the Reference Conditions;
- Accept no claims for unforeseen ground conditions for ground parameters and stratigraphy levels inside the Reference Conditions; and
- Accept no change in ground parameters resulting in an increased strength utilised by the Contractor without being substantiated by a site investigation at least as sophisticated as the Owner's pre-tender investigations.

This approach resulted in the following benefits relating to the tendering as well as the construction phase of the contract:

- The bids returned by each tenderer were based on the same set of ground conditions and on the same interpretation resulting in a uniformity of tenders for tender evaluation and comparison.
- The Contractor's designer could commence his design prior to the results of the site specific ground investigations becoming available.
- Evaluation and approval of the Contractor's preliminary design was made against the Reference Conditions which were formally adopted within the contract framework;
- The details of the pre-tender site investigations provided adequate information to ascertain if the ground conditions were in accordance with the Reference Conditions by relatively simple means.

At the time of writing all site investigations have been finished offshore. The Reference Conditions have been shown to be reasonably accurate with respect to the level of the different strata and on no occasion has the condition of the Limestone been proven to be significantly worse that the parameters in the Reference Conditions.

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FOUNDATION FAILURE OF BRIDGES IN ORISSA - TWO CASE STUDIES

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SYNOPSIS :

The success of an engineer lies in planning, designing & construction of a durable structure. As such every engineer aspires to be crowned with "such success", but unfortunately at times, because of certain factors beyond comprehension, failure does occur. Failure of Bridges may be, either due to failure of superstructure or failure of foundation. In Orissa we have got rare experience of foundation failure. However two case-studies are depicted in this paper.

The first case refers to the famous Tel Bridge near Belgaon in Western Orissa. Originally in this location there used to exist a submersible bridge of total length 340.40 M. In 1967 because of exceptional flood and excessive scour, major part of the bridge was washed away. This necessitated construction of a new bridge, consisting of 12 spans each of 38.26M centre to centre having total length of 459.15M, at 393M down stream of the damaged bridge. By September,1977, when construction of major part of the bridge was completed, the pier well No.2, 3 & 4 alongwith 3 No. of superstructures between well No.5-4, 4-3 & 3-2 were completely washed away, because of another exceptional flood. On evaluation of the causes of failure, it was found that all the wells were plugged at 1.8M to 2.4m below the availability of rock level. Though Safe Bearing Capacity in each case far exceeds the design base pressure, **erodibility of the rock had not been tested**. In reality the foundation strata contains a whitish calcareous material, which leaches out by running water causing partial collapse of the rock strata, resulting in collapse of foundation and super-structure.

The second case refers to Surlake Cut Bridge on the famous Puri-Konark Marine Drive. This is a High Level Bridge of R.C.C. Box Culvert of 7 units of 4.88M centre to centre, with a central suspended span of 2.44M. There is discontinuity at the foundation level as well as deck slab level. Up stream & Down stream are protected by sheet pile. Because of a breach in a nearby river in 1997, its discharge found its way through this small channel & about 10 times the design discharge passed through the box culvert causing 325mm settlement on the Down stream end of the suspended span and bottom raft, leading to abolition of traffic movement on top of it.

1. Introduction :

The term "Failure", though creates despair & despondency in human mind, often leads to interesting findings, when the misty clouds of causes of failures are unravelled. Thus finding out causes of failure always represents an indicator of growth of technology. And hence the importance of highlighting the failure. Foundation failure is one of the most important modes of failures of any type of structure, more so incase of bridges. Two case studies are brought out here only for this purpose.

2. Brief History of Tel Bridge :

Originally the Tel Bridge near Belgaon, opened to traffic in 1957, was a submersible bridge of total length 340.40M consisting of 38 No. clear spans, each of 7.62M & 1 No. clear span of 6.10M. The deck level & Highest Flood Level of the bridge was at RL 165.85M & 170.25M respectively. This bridge was washed away partially during exceptional flood in 1967 because of the following reasons :

a) Trees hitting the bridge.
b) Excessive flood for which the bridge was not designed.
c) Excessive scour uprooting the pile foundation (smaller dia & length).

As this used to be an important link connecting Bolangir & Bhawanipatna, the two district head quarters on State Highway 2, immediately a bridge was planned at 393M Down Stream of the damaged bridge. The new proposed High Level Bridge consists of total length 459.15M with 12 spans each of 38.263M centre to centre; resting over foundation with twin-wells. The work was started in March, 1969 by one agency.

The salient features of the bridge are as follows :

Maxm. Discharge	:	11325 Cumec.
Velocity	:	3.19M/Sec.
Highest Flood Level	:	171.04M RL.
Maxm. scour level	:	146.32M RL.
Foundation level	:	137.20M RL.

During the construction, the high flood of 1973 necessitated construction of one additional span of 15.24M on right side (Kesinga side). By 1973, 8 intermediate wells, 1 abutment well & 7 piers were completed when the first agency left the work. The bridge was started by the second agency in October, 1975. By September, 1977, the major portion of the bridge work was completed except the following components :

i) Sinking of pier well No.1 on right side (Kesinga side) was in progress (well has been numbered from right to left bank).

ii) Casting of superstructure span 11 & 12 were to be taken up (superstructure has been numbered from left to right bank).

Suddenly again on 13.09.77 the Highest Flood Level rose as high as 171.63M RL & the discharge, calculated corresponding to the Highest Flood Level came out to be 15289 Cumec i.e. about 35% excess over the Design discharge of 11325 Cumec. The wells under each pier were twin circular wells each of 4.27M external dia with 0.75M steining thickness. Because of such unprecedented flood, pier well No.2, 3 & 4 towards right side (Kesinga side) along with superstructure span No.8, 9, 10 between well No.5-4, 4-3 & 3-2 respectively were washed away. The rest of the bridge remained in tact, although the river bank of left side (Belgaon side) was eroded considerably both in Up stream & Down stream & was submerged under water. No distinct bank used to exist on this side possibly due to considerable erosion and deposition of thick pile of river born sediments. However the right bank (Kesinga side) is firm with steep bank slope & the tendency of water is to flow in this portion.

3. Causes of failure of Tel Bridge :

After the above disaster, a High Power Committees (HPC) consisting of some eminent engineers of the country was formed to find out the causes of failure. After a careful study of the available records, data & the evidence received from various officers, the committee was of the opinion that the damage to the bridge had occurred on account of -

i) Increased discharge than that for which the bridge was designed.

ii) Concentration of flow with high velocity towards Kesinga side (right side).

iii) Erosion of the strata on which the wells were founded.

The location of the site was not an ideal one as it was just below a constricted section of the river channel resulting in concentration of flow, high velocity & turbulence towards the right bank. However, if the rocky strata on which the foundation was located, was strong enough, even the turbulence and high velocity should not have affected the safety of the structure. This site, however, has the advantage of having the shortest bridge length. It seems that the siting of the bridge was done on the assumption that the rock is hard and inerodible, giving a false sense of safety.

It was found from the records that rock was available at a much higher level than the stipulated founding R.L. In almost all the cases the wells were taken 1.8M to 2.4M inside the rock and then founded.

The rock samples from each well were sent to the laboratory for test. As the Safe Bearing Capacity of rock reported by laboratory was much more than the design base pressure, those were presumed to be suitable for founding the wells. As such the wells were not taken to the stipulated founding level. The table below would explain the same in details. Since each pier is supported below by twin circular wells, the wells are designated in each location as Up Stream(U/S) & Down Stream(D/S) well.

Well No.	Founding RL in metre.	Safe Bearing Capacity (MPa)	Calculated Base Pressure (MPa)
2	U/S 152.91	1.03	1.01
	D/S 152.91	1.03	
3	U/S 152.65	4.30	0.86
	D/S 152.79	2.92	

Well No.	Founding RL in metre.	Safe Bearing Capacity (MPa)	Calculated Base Pressure (MPa)
4	U/S 151.51	4.41	2.34
	D/S 151.51	4.41	
5	U/S 151.17	6.13	2.34
	D/S 151.17	6.13	
6	U/S 151.40	4.98	2.34
	D/S 151.40	4.98	
7	U/S 150.81	8.40	1.96
	D/S 149.70	3.65	
8	U/S 148.37	2.22	1.82
	D/S 148.45	2.22	
9	U/S 138.08	3.45	1.20
	D/S 138.08	3.01	
10	U/S 146.64	7.86	1.97
	D/S 146.00	7.68	
11	U/S 152.44	1.42	1.31
	D/S 152.44	2.99	

The only short coming in the project was that Geological investigation of the rock by an expert Geologist was not conducted which would have revealed a typical nature of the rock. The actual reason of the failure was the founding of the well on a peculiar type of rock formation.

On the advise of the High Power Committee, a Geologist was deputed for the purpose of investigation of rock strata. According to him, the rock strata is dipping at 45^0 to the horizontal sloping towards left bank & Up stream side. The rock layers are intervened with a thick layer of whitish powdered chalky material which get leached out when absorbed in water. Some percentage of white mass which is found within the layers of the rock, rather get disintegrated by coming in contact with water and lead to the collapse of the rock layers. This caused the collapse of foundation & superstructure of 3 spans of the bridge. Hence it may be stated that the rock on which the wells have been founded are partially erodible.

4. Recommendation of High Power Committee(HPC) to protect the undamaged portion:

As the damaged wells were lying along the bridge axis, these could not have been reconstructed at the same location. Therefore the wells were sunk at suitably shifted positions along the bridge axis resulting in change of superstructure spans. The flood protection in the form of garlanding ring with the help of concrete blocks of 1.2M x 1.2M x 1.2M interconnected with 32mm dia hook bars with loose chain for easy launching, both in Up stream & Down stream of the bridge axis were provided to resist the water current of high velocity of the order of 4.27 M/Sec in accordance with the suggestion of the High Power Committee. The cost of such protection work was high. However this type of floor protection was considered essential for the safety of the bridge, particularly for the undamaged spans whose foundations can also be eroded, if the concentration of flow shifted to left side.

It was observed during the flood of 1977 that apart from the damage to 3 spans of the bridge, water spilled on the bank causing a breach of about 122M on the Belgaon side (left side) approach road to the bridge. A suitably designed spill portion was provided as per the suggestion of the High Power Committee on the left side with proper pitching such that provision was made for passing 11325 Cumec discharge through the bridge proper and balance through the spill portion on left side (Belgaon side) and it was made in such a way that in the normal flood, the communication system is not disrupted by more than one or two days.

The bridge was completed by a third agency after many hurdles & finally opened to traffic in the year 1994.

5. Brief History of Surlake Cut Bridge :

Surlake Cut Bridge is located at 10 KM from Puri on the famous Puri-Konark marine drive. It was completed in the year 1982 during the construction of Puri-Konark Marine drive, which was made to complete the Golden Triangle of Orissa consisting of Puri, Konark & Bhubaneswar. The traffic density of foreign tourist is quite high on this road and hence its importance.

Unlike the Tel Bridge, Surlake Cut Bridge is a small, but important High Level Bridge, consisting of R.C.C. box culvert of 7 units of 4.88M Centre to Centre each with a central suspended span of 2.44M. There is discontinuity at the foundation level as well as the deck slab level. Up stream & Down stream end are protected by sheet pile.

6. Causes of Failure of Surlake Cut Bridge :

The bridge was designed for a discharge of 85 Cumecs. Suddenly in the rainy season of 1997, because of a breach in the nearby River, Nuanai, discharge of about 850 Cumecs (i.e. 10 times the design discharge) passed through the box culvert. Because of such exceptionally high discharge, the velocity of water increased to a great extent and it displaced the sheet pile cut-off provided earlier and caused heavy local scour, forming deep gorge to the tune of about 5M in the Down stream end from Left Bank (Konark side) to the centre of the bridge and its depth gradually reduces towards the Up stream. The raft has been cracked & displaced in the central portion of the bridge. Near the joint of the suspended span there is a settlement of about 325mm on Down stream end. Thus the portion of the Box Structure of Left Bank (Konark side) has been tilted towards the Down stream end. As a deep gorge has been created in the Down stream end and the raft has settled to a great extent, it is quite risky to jack up the superstructure to restore the relative settlement of 325mm of the superstructure. A Baily Bridge type arrangement was made to restore the traffic on such an important route.

7. Conclusion :

Fixing of the founding level in case of bridge depends on the discharge, waterway provided and type of strata of the river bed. For different types of soil the maximum scour level & founding level can be calculated depending on the silt factor, flow concentration & water way etc.. But in case a sound hard inerodible rock is encountered at a level, higher than the Maximum Scour Level, then the scour line is considered to be the top of the rock level and founding level can be fixed by keeping the structure below the rock with some minimum grip length as per relevant code. But to take a decision, whether a rock is hard & inerodible, is a tricky one. In such cases, it is always preferable to take the help of an experienced geologist to find out the nature of rock and the rock profile (Dip & fault), along with field testing of the quality of rock (both Safe Bearing Capacity & erodibility). Then only the decision on foundation can be safe & suitable.

ACKNOWLEDGEMENT

The authors are grateful to the authorities of the Works Department, Government of Orissa, for allowing to see the different relevant documents on the above two bridges and permit to publish this paper.

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CONSTRUCTION AND MAINTENANCE PROBLEMS OF KALIABHOMARA BRIDGE ACROSS RIVER BRAHAMPUTRA NEAR TEZPUR ON NH-37A

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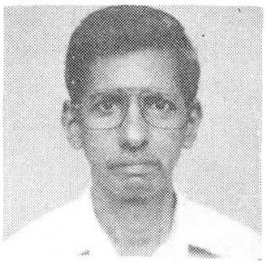

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SUMMARY

The Kaliabhomara Road Bridge across Brahmaputra near Tezpur, Assam (India) has 24 main spans 120.00m and two shore spans 67.50m each. The span arrangement consists of Pre stressed Concrete balanced cantilever arms of 52.50m long on either side of piers with a central Reinforced Cement Concrete suspended span of 15.00m.

During the construction of the bridge certain problems were faced and the remedial measures taken were as under: -

1) Compulsion of plugging of one well much above the design founding level solved by providing 5 No. bored 1.50 m R.C.C. piles under the well cap and a wire crated boulder garland 60 m dia, 3m thick to check scour.
2) Encountering part rock and part soil at open foundation level of other pier solved by providing anchor rods in the rocky portion and anchoring the portion on the soil by providing piles.

The problems faced during maintenance and remedial measures taken are as under: -

1) Shift of main channel at the location of well which was plugged much above the designed founding level, washing away the wire crated boulders garland provided at the time of original construction, tackled by replenishing the same upto RL 45.00m as per maintenance manual guidelines.
2) Tilt of another well due to a series of earthquake causing damage to the slab seals of the elastomeric slab seal type of expansion joints, being replaced as and when required as a temporary measure. The permanent measure being replacing the expansion joints by strip seal/modular type of expansion joints now available.

BRIEF PARTICULARS OF THE BRIDGE

The Kaliabhomara Bridge near Tezpur has 24 main spans 120.00m and two shore spans 67.50m each. It was completed in the year 1987. The span arrangement consists of balanced cantilever arms of 52.50m long on either side of piers with a central suspended span of 15.00m long. The total length of the bridge is 3015m. The cantilever arms are of Prestressed Concrete Box girders over the pier and the suspended span consists of two R.C.C.girders jointed by three cross diaphragms and the deck slab. One abutment of the bridge is located on the Bhomorgauri hill on right bank (North side) and the other abutment is located near the village Burbandha Chapori on the left bank (south side). A guide bund is provided on the south bank. A schematic view of the bridge is shown in Figure 1. The bridge is located in seismic zone V of the country is seen to be subjected to shaking often due to small earthquakes.

During the construction of the bridge certain problems were encountered for the foundation work. Also after completion of the bridge some problems of maintenance were faced mainly due to subsequent floods and a series of earthquakes which took place in the year 1988,1990. Particulars of problems and the remedial measures are detailed below:

PROBLEM NO.1

Sinking of well under Pier P2 was the first activity with which the work of Kaliabhomara Bridge commenced. It also so happened that it was the last foundation to be completed. As per preliminary sub-soil investigation report the strata for founding this well appeared to have dense sand. However after sinking for 25m deep upto RL 35.60m against the design founding level of RL 5.00m, the work slowed down as it encountered bouldery strata. For two seasons the work of removal of boulders continued and after removing 100 boulders the well reached a level of RL 32.08m. At this point the stability of the well was checked for scour considerations and it was established by the calculations that if the scour could be controlled at RL 45.00m the well will be safe. Secondly though the calculations proved that the well would be safe under normal conditions, under seismic effect there was tendency to develop tension.

REMEDIAL MEASURES

To arrest scour at RL 45.00m it was decided to provide a boulder apron around the pier. This was feasible since the well was near the bank. The well was protected by providing 60.00m dia, 3.00m thick crated boulder garland packed under dry conditions prevailing at the time of construction of the bridge at the location of well under pier P2 (Figure 3). The top level of this garland was kept as RL 64.12 and it was decided that the performance of this garland should be closely monitored so that under no circumstances the scour be allowed to reach below the RL 45.00, the calculated maximum scour level for which the well was found to be safe. Regular probing is being done now to see if the level has gone below RL 45.00 in which case crated boulders are dumped to make up the level difference.

To solve the second difficulty of development of tension under seismic conditions, it was decided to provide 4 No.R.C.C.bored piles of 1.50m dia on the outer periphery outside the well and 1 No. similar pile in the middle though the dredge hole (Figure 2). Apart from this a combined well-pile cap, square in size was provided so as to act as a composite unit. This was based on the analogy of stays for a tower. The piles however had to be terminated at different levels due to the rock sloping steeply towards the riverside. The central pile has been taken to RL 24.00m and other piles to levels varying from RL 29.00 TO RL 32.00m. Thus the piles take care of the tensile (uplift) forces under seismic condition and ensure stability of the foundation as a whole. The above solution was preferred since the other alternative of anchoring well by inserting anchor rods through well steining or driving piles through the bottom plug was not considered satisfactory due to bouldery strata.

PROBLEM NO.2

The bridge work was taken up in December 1981 and completed in all respects by March 1987. When the bridge was under construction the river bed near pier P2 was dry. Later on during the floods of year 1991, the river started attacking the north bank and active channel developed near P2. The wire crated boulder garland was washed away and scour around this well extended up to RL 37.47.

REMEDIAL MEASURES

Protection measures and replenishment of washed away boulders were carried out by dumping wire crated boulders and the top level of the garland was brought back to RL 45.00. A close monitoring of the top level of the garland up to RL 45.00 is being maintained by taking levels at regular intervals. This level has appeared to stabilize at RL 45.00.

PROBLEM NO.3

During the year 1988 in the month of April and August a series of earthquakes in the region took place. A routine inspection of the bridge was carried out after the first earthquake on April 16-17/ 1988 when it was noticed that pier P3 showed a tendency of tilting towards south. Further routine inspections were continued and the following inferences were drawn: -

1) During the period from April 1988 to September 1988 deck at the location of pier P3 towards south has increased from 55mm to 83mm i.e. 28mm.

2) The expansion gap in P3-P4 span on u/s side was more than that on the d/s side at the fixed end, indicating that the Pier P3 has a tendency to tilt towards south west direction.
3) The elastomer pad slab seal type expansion joints for the suspended spans for P2-P3 and P3-P4 were under sustained shear stress and there had been same related movement between top and bottom plates of the knuckle-cum-elastomeric/knuckle-cum-sliding type of bearings with seismic arresters.

4) The railings on the u/s side at fixed end had touched each other, which also indicated tendency for tilt.

POSSIBLE CAUSES OF TILT OF PIER P3

1) Although the movement of P3 was first noticed in April 1988, it is to be stated that the slow movement occurring much before that date can not be ruled out. The effect of movement was noticeable by the visible distress in expansion joint pads &

reduction or increase of expansion gaps. The expansion joint pads were placed in March 1987 and the damage was noticed in April 1988 just about a year after placing the pads. Therefore if any movement which could have occurred prior to placing of pads this cannot be determined as there are no records of visible movement.

2) From the bore log for foundation of pier P3 it could be noted that the well was not seated on rock. The well is resting on 2-3m thick layer of pebbles/cobbles mixed with sand and therefore movement due to unevenness of bearing capacity could be possible.
3) It was possible that the repeated shaking due to the earthquakes could have triggered of the movement.
4) In Jan 1990 another earthquake took place when the number of shocks were seven. However this didn't cause any further movement. This showed that pier P3 appeared to have stabilized because of large passive resistance built up due to earlier movement in foundation.

REMEDIAL MEASURES

1) The slab seals of the expansion joints are being replaced.

2) The pier P3 is kept under constant observation and as already mentioned in (d) in prepara no further tilt is noticed.

3) A garland of wire crated apron for a thickness of 2.50m has been provided with the top at RL 44.50m and for a diameter of 66m.

SECONDARY EFFECT DUE TO THE PROBLEM NO.3

The clear gaps provided between faces of cantilever end and suspended spans are 40mm and 110mm at fixed end and free ends respectively. On the advice of Prof. Leonhardt elastomeric slab seal type of expansion joints were provided. However the expansion joints provided at free end are capable of catering for a movement of +/-62.50mm. The technological development at that time in this respect suggested that wider expansion joints would have been far more costlier and it was thought the utility of these joints would have been called into play only in most sever combination of movements. However these extreme combinations of movements have probably come into play because of the severity of earthquakes and it is observed that the seals of expansion joints are getting damaged frequently.

REMEDIAL MEASURES

There is no other quick alternative to the above problem than that replacing the damaged slab seals at regular intervals. The only other solution is the replacement of the expansion joints for the entire bridge by providing the latest single strip/modular type of expansion joints at fixed/free ends, which however can probably wait for the time being.

PROBLEM NO.4

During initial sub-soil investigations conducted in 1980 it was observed that the founding strata for pier well P1 would be on a dipping surface of rock. To overcome this the bridge was shifted by about 60m towards northwards. However when detailed boring was conducted at the shifted location of P1, it indicated the presence of a combination of hard strata in northern half at a higher location and soft strata in southern half. On practical considerations, it was clear that it was not possible to provide a well foundation for shallow depth due to risk of tilting. Thus there was a design problem.

REMEDIAL MEASURES

A separate design for pier P1 was carried out by providing a raft foundation for supporting the pier. The raft was designed in the following way: -

1) On the south side where the strata was soft, the raft was supported on 6 No. of bored piles of 1.50m dia.
2) On the north side where hard strata was found, the raft was anchored into the rock by means of 98 Nos.36 mm dia HYSD rock anchors.

Further in the design it was assumed that the piles would not share any longitudinal force. However the rock anchors would share part of the longitudinal force. They were designed for 9 t of uplift force. Balance force was proposed to be taken care of by providing shear keys underneath the raft in the form of beams embedded in the strata below. For this purpose large size R.C.C. shear keys were provided in both directions under the pile cap (raft) by cutting trenches below soil in a grid pattern and casting beams which formed an integral part of the raft.

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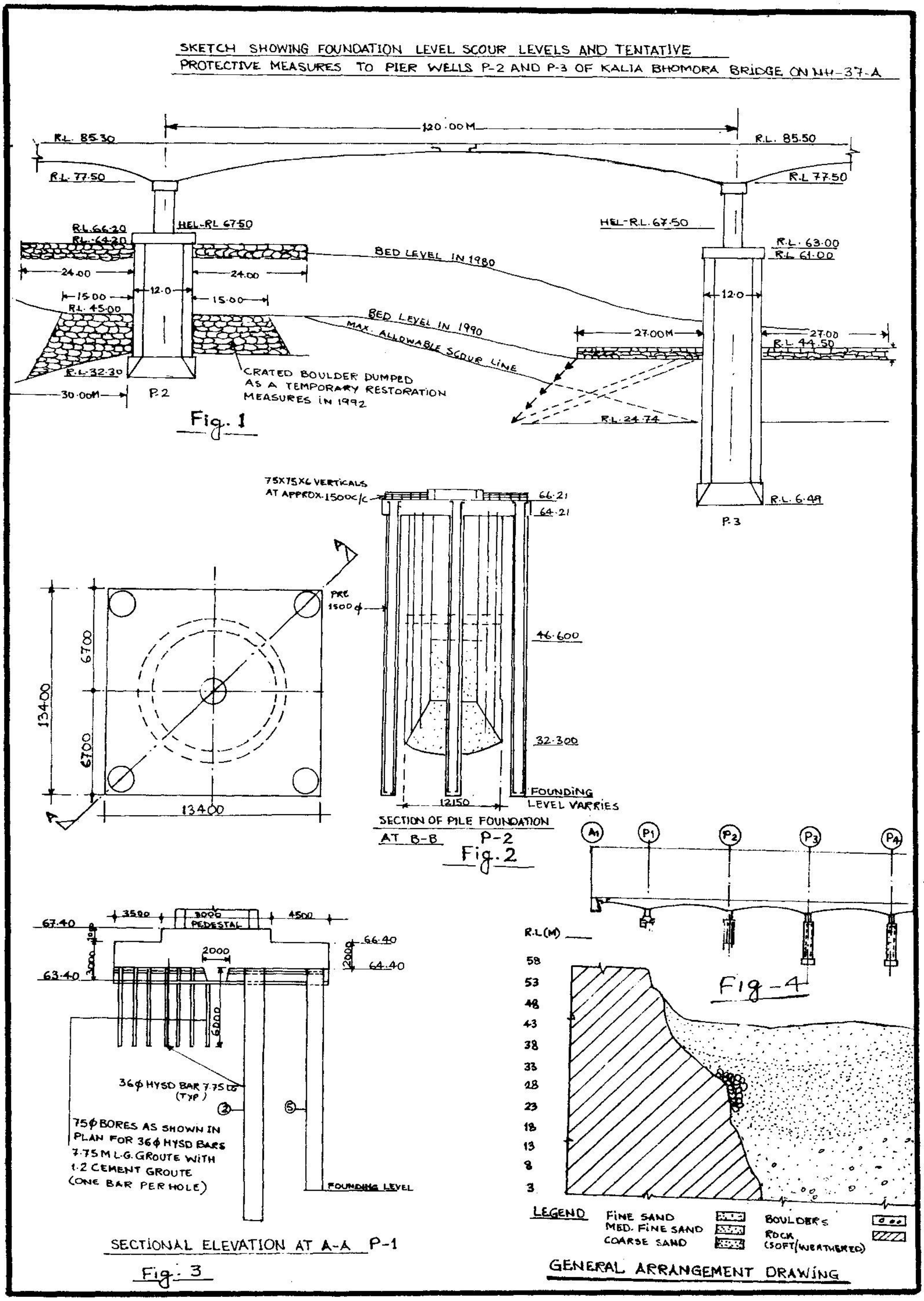
SKETCH SHOWING FOUNDATION LEVEL SCOUR LEVELS AND TENTATIVE PROTECTIVE MEASURES TO PIER WELLS P-2 AND P-3 OF KALIA BHOMORA BRIDGE ON NH-37-A
120.00M
R.L. 85.30
R.L. 85.50
R.L. 77.50
R.L 77.50
HFL-RL 67.50
R.L.66.20
R.L.64.20
R.L. 63.00
R.L. 61.00
BED LEVEL IN 1980
BED LEVEL IN 1990
MAX. ALLOWABLE SCOUR LINE
CRATED BOULDER DUMPED AS A TEMPORARY RESTORATION MEASURES IN 1992
R.L. 45.00
R.L. 44.50
R.L.32.30
R.L. 24.74
R.L. 6.49
P.2
P.3
Fig. 1
75X75X6 VERTICALS AT APPROX.1500c/c
PRE 1500φ
66.21
64.21
46.600
32.300
FOUNDING LEVEL VARRIES
SECTION OF PILE FOUNDATION AT B-B P-2
Fig. 2
13400
6700
6700
12150
SECTIONAL ELEVATION AT A-A P-1
Fig. 3
PEDESTAL
67.40
63.40
66.40
64.40
36φ HYSD BAR 7.75 LG (TYP)
75φ BORES AS SHOWN IN PLAN FOR 36φ HYSD BARS 7.75 M L.G. GROUTE WITH 1:2 CEMENT GROUTE (ONE BAR PER HOLE)
FOUNDING LEVEL
A1 P1 P2 P3 P4
Fig-4
R.L(M)
58 53 48 43 38 33 28 23 18 13 8 3
LEGEND
FINE SAND
MED. FINE SAND
COARSE SAND
BOULDERS
ROCK (SOFT/WEATHERED)
GENERAL ARRANGEMENT DRAWING

INNOVATIVE BRIDGE FOUNDATION FOR HILLY REGIONS

*DEENA NATH

Summary

The CIDF system is a combination of a group of Auger Bored Compaction Under-reamed Piles (ABCUP) of small length with grid beams and top slab supported on soil may be used as an innovative foundation system for bridges in eastern provinces of India. However this paper deals with analysis of the foundation and provides some useful data required for design in the field.

Introduction

The innovative design of combined intermediate depth foundation (CIDF) system is derived from the stilt root system of plants. In present era of development demand of energy is increasing day by day resulting in effort requirement for the digging of more and more oil wells. presently we are looking for north-east provinces of the country for the natural source of energy. These provinces are mostly surrounded by the hills. For the transportation of oil and gases piplines are used and for crossing the deep gorges cable bridges are frequently used. However this paper is devoted to the analysis of suitable bridge foundation especially for these regions.

The Bridge Foundation

A modified structural form comprising of conventional under-reamed piles (ABCUP) combined with grillage cap for transferring the load of bridge particularly at the banks of stream. As we know the deep gorges and valleys are the common feature of hills. The CIDF system seem to be very suitable where load transferring is not possible at the shallow depth level and anchoring is required. The essential requirements in the design of a foundation are

i. The total settlement of the structure
ii. Differential settlement along with other design capabilities.

To limit settlements it is necessary to transfer the load of the structure to the soil stratum of sufficient strength and to spread the load over a sufficiently large area of the stratum to minimise bearing pressure. However, if the soil of adequate strength is not available immeiately below the bridge support CIDF system (Fig. 1 and Fig.4) may be used to transmit the load.

The ABCUP

The range of the allowable skin friction for auger Bored Compaction Under-reamed Pile (ABCUP) varies between 180 to 300 kN/m2 (Brandl 1988) where in ordinary under-reamed pile maximum value of skin friction is only 110 kN/m2. still more allowable values of the skin friction are expected but for the design purposes these values are suggested for evaluating the effectiveness of CIDF system.

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Foundation and Natural Root System

To extend the correlation and draw useful inference between natural plant root system and the man made foundation system, overall study of various rott system, particularly the modified form are logical. Similar to foundation the major function of root is to anchor the plant and resist load due to weight, wind and so on. Functional comparison of foundation and root further reflect similarity in mechanism of load transfer in both the cases friction and by bearing resistance.

Skin Friction

Apart from vertical load transmitted to soil mass by skin friction at side faces of capping beam it also transmit the load in bearing for design purpose of well foundation, the values of skin friction as recommended by Terzaghi and Peek (2) given in Table-1 may be used for load transfer contribution.

Table - 1
Recommended Skin Friction

Type of soil	Density kN/m^3	Skin Friction kN/m^2
Silt & soft clay	130	7.25-30
Very stiff clay	143	50-200
Loose Sand	151	10
Medium Dense sand	181	15
Dense Sand	212	50-100
Dense gravel	212	50-100

Assumption

Following assumptions are made for composite block analysis.

1. The depth of the block is considered equal to the sum of depth of beam and the length of pile.
2. The total axial lond carrying capacity of the block is equal to the sum of resistance offered by the all four sides of the foundation block due to allowable skin friction and allowable bearing offered at the face of the block. (Fig.1)
3. Due to confinement and densification of soil the block, coefficient k_s improves so that it is more realistic to assume improved of the skin friction for calculation of sides resistance of the block.
4. The combined structure of slab and beam will jointly behave like a rigid cap.
5. The soil contained within the peripherally circumscribing all the piles behave like a solid composite confined mass.

3.4.2 Assumption for CIDF system as small group of piles.

Following Assumption are made for CEDF system as small group of piles.

1. It is assumed that all piles settle equally and bear same load. for ABCUP the allowable skin friction is given in Table 1.
2. Slab and beam are softer than the piles so only a fraction of the allowable bearing pressure at the level of slab and beam will be mobilised. For this analysis this fraction has been assumed as one half the value given in Table -1.

Analysis

The analysis of foundation may be made on the basis of considering as a rigid disk (Fig. 2a and Fig.2b) at distance 'b' deflection is zero.

Small group of 9 piles displacement as rigid disk in soil annulus.

So that vertical deflection is given by

$$\frac{d^2w}{dl^2} + \frac{1}{r}\frac{dw}{dr} = 0 \qquad (1)$$

when the displacement of the disk is w_0 the solution of the above equation may be given by

$$w = w0 \left[1 - \frac{\ln r/d}{\ln b/d} \right] \qquad (2)$$

The vertical load from the bridge applied through the grillage slab cause all piles to deflect through the same vertical distance. The deflection at a point at radius r/d, w (r/d) due to a load F applied to a rigid centrally located disk radius d is

$$w\,(r/d) = f/2\pi G \ln \{ (b/d)/(r/d) \} \qquad (3)$$

if r/d = b/d then w (r/d) will be zero.

Depending upon the load 9p or 5p system can be used.

$$F0 + 4F1 + 4F5 = F \qquad (4)$$

After normalization by F the above equation can be rewritten as

$$f0 + 4f1 + 4f5 = 1 \text{ where } f0 = F0/F \qquad (5)$$

from Eq. (3) we see that the deflection of the three typical pile sections are

$$w0 = \frac{1}{2\pi G}\left[f_0 \ln\left[\frac{b/d}{1}\right] + 4 f_1 \ln\left[\frac{b/d}{2s}\right] + 4 f5 \ln\left[\frac{b/d}{2\sqrt{2s}}\right]\right] \qquad (6)$$

$$w1 = \frac{1}{2\pi G}\left[f_1 \ln\left[\frac{b/d}{1}\right] + f_0 \ln\left[\frac{b/d}{2s}\right] + 2 f_1 \ln\left[\frac{b/d}{2s}\right] + 2 f1 \left[\frac{b/d}{2\sqrt{2s}}\right] + 2 f_5 \ln\left[\frac{b/d}{2\sqrt{5s}}\right] + f1 \ln\left[\frac{b/d}{4s}\right] \qquad (7)$$

$$w5 = \frac{1}{2\pi G}\left[f5 \ln\left[\frac{b/d}{1}\right] + 2f1 \ln\left[\frac{b/d}{2s}\right] + f0 \ln\left[\frac{b/d}{2\sqrt{2s}}\right] + 2f5 \ln\left[\frac{b/d}{4s}\right] + 2f1 \ln\left[\frac{b/d}{2\sqrt{5s}}\right] + f5 \ln\left[\frac{b/d}{4\sqrt{5s}}\right]\right]$$

But since these deflection must all be the same we can write

$w0 - w1 = 0$ (9)

and $w0 - w5 = 0$ (10)

The eq. (9) and Eq. (10) can be writen as

$f0 \ln 2s + f1 \text{ in } (2/s) + f5 \ln (5/4) = 0$ (11)

and $f0 \text{ in } 2\ 2s + f1 \ln 5 + f5 \ln [(2)/5] = 0$ (12)

from Eqs. (5). (11) and (12) one can get f0. f1 and f5 for the value of s selected. When a solution is obtained. the dimensionless displacement $w' = 2 \Pi GW/F$ of the pile disk group can be find out by the use of equation (5) and (6) or (7).

The subgrade reaction coefficient for each pile is computed by dividing the dimensionless force in the pile by displacement w' thus

$Ki = 2\Pi G\, fi / w'$ (13)

However the coefficent K for a solitary pile can be written

$K = 2\Pi G / \ln (b/d)$ (14)

The ratio of the individual reaction coefficient can be obtained

$Ki / K = fi / w' \ln (b/d)$ (15)

The relevant values are given in Table 2.

Table - 2

Displacement. Forces and Relative Stiffness in a 9p system

Spacing Diameters, S	F0	F1	F5	w'	Group Displacement Efficiency k_n/k or	k_0/k	k_1/k	k_5/k
2	-0.0298	0.0721	0.1854	2.2274	0.195	-0.0524	0.1266	0.3256
3	-0.0024	0.0820	0.1686	1.8822	0.231	-0.0051	0.1705	0.3504
4	0.0116	0.0864	0.1607	1.6338	0.266	0.277	0.2069	0.3848
5	0.0203	0.0890	0.1560	1.4399	0.302	0.0551	0.2417	0.4237
6	0.0264	0.0907	0.1527	1.2810	0.339	0.0806	0.2770	0.4663
8	0.0345	0.0929	0.1484	1.0294	0.422	0.1312	0.3697	0.5641
10	0.0417	0.0973	0.1423	0.8439	0.515	0.1932	0.4510	0.6596

Allowable Load Carrying Capacity of CIDF System

(a) Block Foundation (Fig. 1)

P = [Fs + Fb] - - weight of the block

Where Fs and Fb are average allowable frictional resistance and allowable bearing resistance at Depth 'D' and P is the load carrying capacity

(b)CIDF system as a small group of piles.

P = Pp + Pb + Ps (17)

Where Pp, Pb and Ps are the load carried by the ABCUP, beam and the slab individually. (Fig.4)

However the load carrying capacity of the foundation may be computed by the use of Eq. (16) and Eq. (17) and the lower one will be treated as the load carrying capacity of the foundation. the load settlement and other values regarding load, spacing and diameter of ABCUP are shown in fig. 3, Fig.6 and Fig.7

Conclusion

Following are the main conclusions

1. If pile spacing is up to three times of the diameter, thecentral pile will not be effective so that the minimum spacing of ABCUP should be 4d.

2. When the spacing of ABCUP is more the load carrying capacity of individual pile group will be more so that the whole system, will carry more load. Although, due to interaction effect of pile to pile average load carrying capacity of individual pile in system will be lesser than single pile. But in the CIDF system contribution of grid cap, effect of densification, confinement will also come into the picture resulting in more load carrying capacity of the system.

3. The confining effect in loose soil is more than that of the denser soil.

4. Load carrying capacity must be verified with field tests of minimum 2% of total number of ABCUP used.

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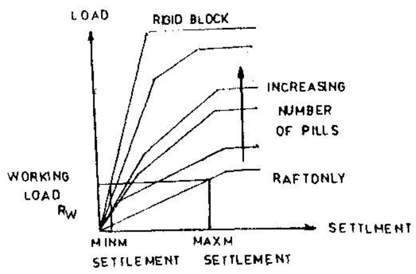

FIG-5 LOAD SETTLEMENT CHARACTERISTICS OF PILE COMBINED WITH CAP AFTER POLOUS AND DEVIS atal. (1972)

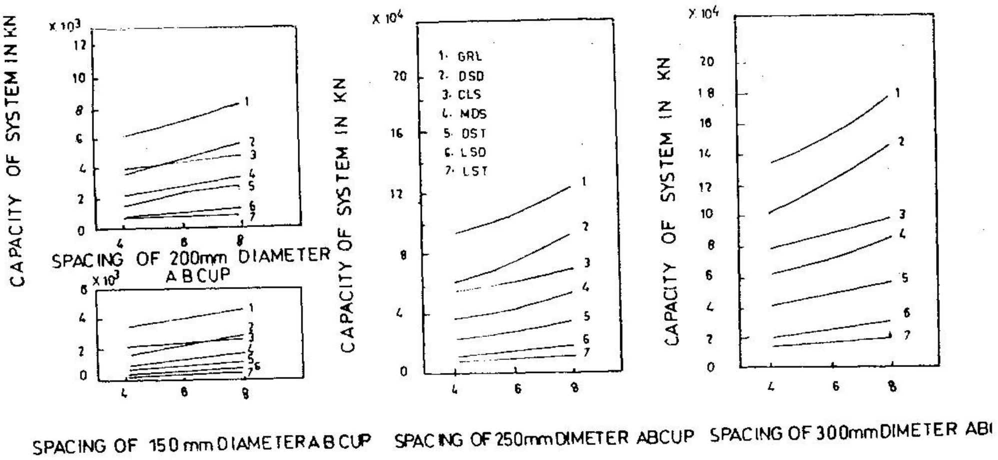

FIG. 6 SPACING Vs CAPACITY OF CIDF SYSTEM

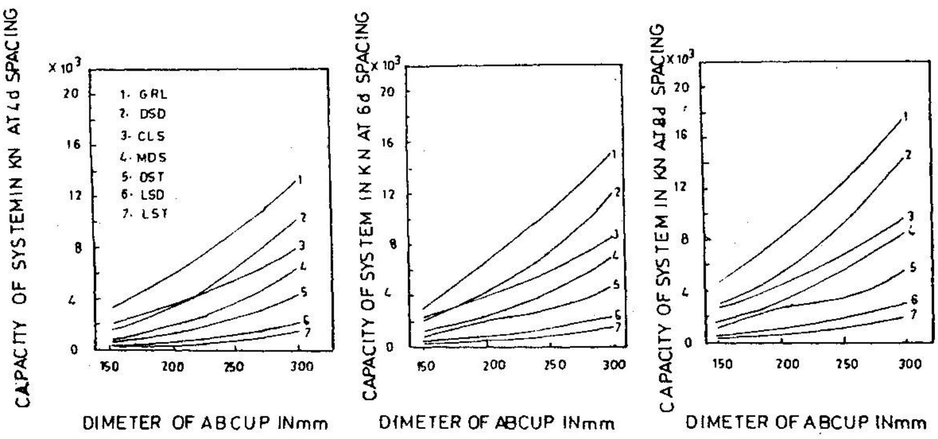

FIG.7 DIAMETER OF ABCUP Vs CAPACITY OF CIDF SYSTEM AT VARIOUS SPACINGS OF ABCUP

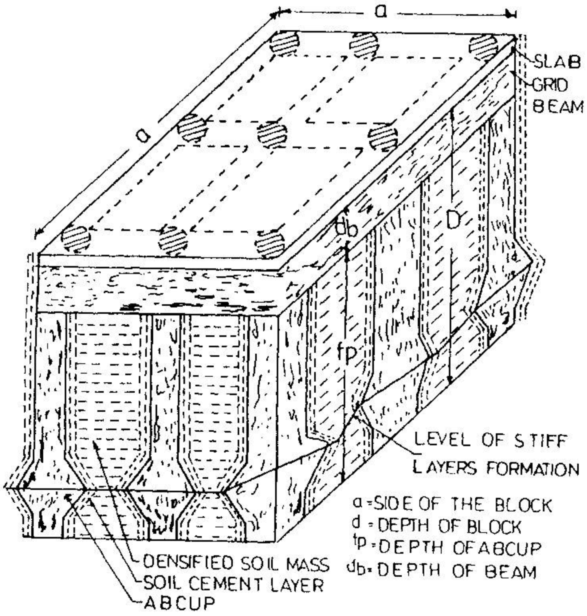

FIG1 CIDF SYSTEM AS BLOCK FOUNDATION

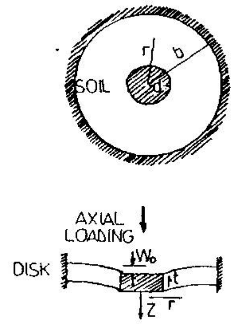

FIG-2a. PILE SECTION DISPLACEMENT AS RIGID DISK IN SOIL ANNULUS

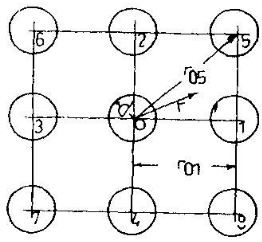

FIG-2b CIDF SYSTEM AS GROUPS OF PILES

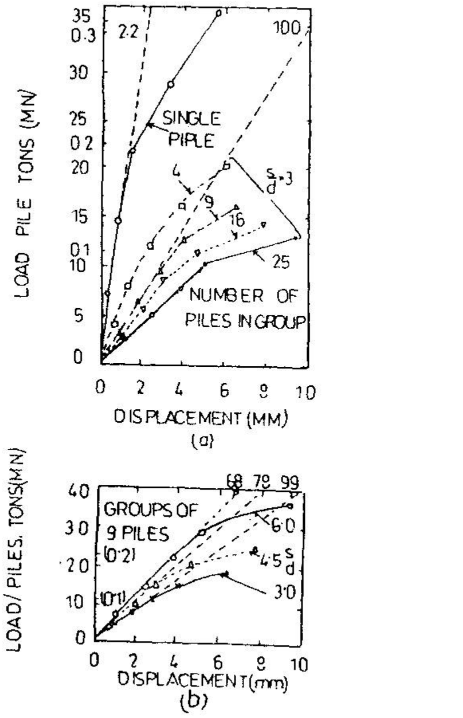

FIG-3. DISPLACEMENT VERSUS PILE LOAD (a) SINGLE PILE & VARIOUS GROUP AT 3d SPACING IN SILTY SAND, (b) NINE PILE GROUP AT VARIOUS SPACING

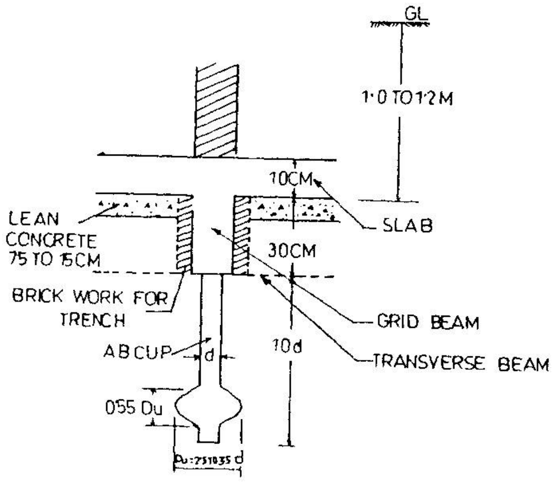

FIG4-A VIEW OF JUNCTION OF BEAM SLAB & ABCUP

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SPECIAL CHEMICALS FOR EFFECTIVE RESTORATION OF DISTRESSED BRIDGE FOUNDATIONS

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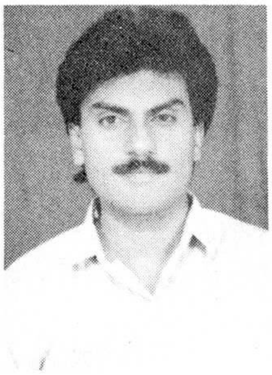

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SUMMARY

Bridges are one of the oldest structures built by man and even now they play a vital role in the development of the country by forming an important part of the infrastructure for both roadways & railways. Many RC bridges constructed in the recent past have begun to show distress features like cracking, corrosion of reinforcement , spalling of concrete. etc. There distress features could be observed in any of the bridge components.

The conventional methods of repair do not provide long term solutions. The newly developed specialty chemicals like polymers, epoxies, micro concretes, polyethanes, etc. help in not only restoring the distressed bridges but will also provide a long term protection against various harmful agent like chlorides, sulphates, carbon dioxide, moisture etc.

This paper briefly discusses the causes for deterioration of bridge structures. The different distress features normally observed in the bridge foundations & the role of special chemicals in effective restoration of the distressed bridge foundations is also discussed.

1.0 INTRODUCTION

Bridges from an important constituent of the transportation segment and millions of Rupees are spent annually not only in the construction of new bridges but also on the repair & restoration of distressed bridges.

A bridge eventually consists of three major structural components namely the foundation, substructure & superstructure.

The distress features could be observed in any or all the units of these components.

2.0 BRIDGE DETERIORATION

Bridge Deterioration may arise from a number of independent sources. These sources may be classified as

a Design and construction deficiencies
b Environmental effects and
c Changes in use

2.1 Design and construction deficiencies : These include

- Insufficient concrete cover over reinforcement
- Grouping in tendons
- Improper compaction of concrete
- Inefficient bearings
- Differential movement/ shifting
- Inadequate spacing between tendons
- Incomplete grouting of tendons
- Bad detailing of reinforcement leading to congestion
- Bad deck drainage system
- Inadequate curing

2.2 Environmental effects : These include

- Material quality
- Environmental aggression (Chloride)
- Freeze thaw deterioration
- Support movement
- Carbonation
- Alkali silica reaction
- Shrinkage
- Thermal strains
- Stress corrosion
- Hydrogen Embrittlement
- Corrosion

2.3 Changes in use : This is a significant factor affecting the bridge deterioration and includes.

- Increase in traffic volume
- Increase in maximum permitted vehicle size i.e. load
- Increase in the number of frequency of large six wheeled vehicles i.e., repeated loading to maximum.
- Wear and fatigue are two mechanisms that directly lead to deterioration of the bridge and its deck.

3.0 DISTRESS IN BRIDGE FOUNDATIONS

The commonly noticed problems in bridge foundations are

- Excessive scouring
- Erosion & cavitation damages
- Settlement of foundation
- Tilting
- Differential movement / shifting
- Damages to protection work
- Corrosion & cracking
- Air voids & Honeycombs

4.0 ROLE OF NEW MATERIALS / SPECIAL CHEMICALS

The conventional methods of repair like patching up mortar/ concrete, guniting, shotcreting, etc. indeed provide a solution to the problem but these are only short term approaches. The conventional methods do not provide a long term solution. It has been found from experience that many bridges repaired with the conventional methods fulfill the serviceibility needs for a short duration of time only and within a few years the structures start developing the same types of symptoms or distresses as was observed before.

The latest specialty repair chemicals developed in the recent times play a significant role in the rehabilitation of the distressed rc bridges. These chemicals not only help in quick and effective repair but also provide long term solutions. The role of these specialty repair chemicals in effective rehabilitation of common types of distress normally observed in bridge foundation is highlighted in Table 1.

5.0 RESTORATION MEASURES

5.1 Excessive scouring

This is a common occurrence in flowing rivers. The method of restoration is to reinstate the worn out concrete with free flow anti wash out high strength cementitious underwater repair micro concrete after cleaning the exposed reinforcement to remove rust & providing with an anticorrosive treatment.

In order to protect from further scouring in future boulders can be dumped around the region and the gap between the boulders grouted with 1:3 cement sand slurry modified with plasticising cum expanding additive.

5.2 Erosion & cavitation damages

The eroded portion can be reinstated with underwater high strength micro concrete as explained in 5.1 above and the area protected by dumping boulders.

5.3 Settlement of foundation

Whenever settlement is noticed in foundation of piers and abutments great care shall be taken to keep the sitution under control and it shall be regularly monitored. The levels shall be frequently checked and the difference be made up with plates or by micro concrete padding. If the settlement is due to scouring then the scoured portion shall be reinstated with free flow underwater micro concrete. Pitching / dumping of boulders around the piers or abutments shall also be done for protection. In case of integrated wing walls settlement is a common feature and it is preferable to build a separate wing wall.

5.4 Tilting

The tilting or leaning is due to excessive pressure or eccentricity of loading & various other factors.

Tilting can be handled by strengthening the existing foundation of abutment or piers by adding extra section. Jacketing shall be resorted and the extra section shall be provided by using high strength cementitious anti wash out underwater repair micro concrete.

5.5 Cracking on surface of foundation

This is a common problem in most of the bridge foundations. The restoration shall be carried out by sealing the cracks externally with underwater epoxy putty and grouting neat cement slurry modified with plasticising cum expanding additive in case of wider cracks (>5 mm) The minute cracks shall be sealed by injecting low viscosity, high strength moisture insensitive epoxy after sealing the cracks from the exterior using under water epoxy putty.

5.6 Corrosion, spalling & delamination

This is also a common problem noticed in the bridge foundations.

This distress can be addressed by providing an anticorrosive treatment to reinforcement after cleaning & reinstating the damaged / delaminated concrete with high strength cementitious anti worn out underwater micro concrete or with water resistant epoxy mortars in case of damage at isolated locations.

5.7 Voids & Honeycombs

This is a common problem in heavily loaded foundations due to congestion of reinforcement. It may also be caused due to poor site construction practices like improper compaction. This can be rectified by pressure grouting neat cement slurry modified with plasticising cum expanding additive.

The various chemicals to be used for different types of foundation distress is also presented in Table 1

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Table 1 : SPECIAL CHEMICALS FOR REPAIRS

Sl.No.	Types of Distress	Repair chemicals to be used
1	Excessive scouring	Anticorrosive coating for exposed reinforcement Anti washout underwater micro concrete for reinstating the scoured portions.
2	Erosion & Cavitation	Anticorrosive coating for exposed reinforcement Anti washout underwater micro concrete for reinstating the scoured portions.
3	Settlement of foundation	Underwater micro concrete for reinstating scoured / damaged portions.
4	Tilting	Underwater micro concrete for jacketing / strengthening of foundation.
5	Corrosion spalling & delamination	Anticorrosive coating for exposed reinforcement. Reinstating spalled areas with underwater micro concrete or water insensitive epoxy mortar.
6	Cracking	Underwater epoxy putty for sealing the cracks. Injection of cement slurry modified with non shrinking grouting additive or injecting low viscosity water resistant epoxy into the cracks.
7	Voids & honeycombs	Grouting with cement slurry modified with placticising , non shrinking additive.

CONCLUSION

The different types of distress noticed in the bridge foundation along with a brief methodology of repair using special repair chemicals has been presented. These specialty Chemicals help in effective restoration & provide long term solutions

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THE CONFEDERATION BRIDGE, CANADA

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SUMMARY

The Confederation Bridge is located in Atlantic Canada; it provides a fixed link across the Northumberland Strait between Cape Tormentine, New Brunswick, and Borden, Prince Edward Island. The design service life of the structure is 100 years. The 13-km Crossing comprises approaches with 93-m spans in shallow water near shores and a main bridge with 250-m spans in the Strait. Because of the short construction time and the often adverse conditions for work at sea, precasting was used systematically on a large scale for the entire bridge. Precast pier bases were installed and grouted to bedrock at depths down to 38 m below sea level. Precast shafts were erected upon the bases. Typical cantilevers for marine spans weighing 78 MN were precast on shore and set in place with a floating heavy-lift crane, which was also used for placement of 52-m- long precast drop-in spans between cantilevers using a procedure which eliminates excessive erection moments in the piers. Innovative design features and the most advanced construction techniques and skills have been called on to match the challenge presented by such major undertaking.

1. INTRODUCTION

A fixed link between the Provinces of New Brunswick and Prince Edward Island had been considered for nearly a century. It was not until 1988, however, that the idea materialized into a 13-km bridge, the Northumberland Strait Crossing. The bridge would be financed, built, and operated by a private developer for thirty-five years, then turned over to Public Works Canada. In 1992 three international joint ventures were on the final list for consideration. Strait Crossing Joint Venture (SCJV), the successful low bidder, is composed of SCI of Canada, Morrison Knudsen of the United States, GTM of France, and Ballast Nedam of the Netherlands. The bridge design consultant is J. Muller International·Stanley Joint Venture. Construction started in Spring 1994 and lasted until the end of 1996, with opening of the facility in June 1997. Considering the size of the project, the short construction time, and the adverse conditions generally encountered at sea in this region, the realization of this bridge was a monumental work.

2. DESCRIPTION OF THE PROJECT

2.1 General

Deep water in the Strait calls for long spans, whereas shallow water near shores is more suited for shorter spans. The extremely short completion time called for massive use of precasting, large precast elements carried by heavy marine equipment for the long spans and more conventional elements for the shorter spans. These requirements naturally divided the bridge into two different structures: the long marine spans in deep water, and the shorter approach spans in waters not accessible by deep draft vessels.

From Jourimain Island on the New Brunswick side to the village of Borden on the Prince Edward Island side, the bridge comprises:

- The West Approach, 1,320 m long with 93-m spans.
- The Main Bridge across the Strait, 10,990 m long with 250-m spans.
- The East Approach, 600 m long with 93-m spans.

2.2 Main Bridge Substructure

2.2.1 Foundations

The rock sequence across the Strait consists of a series of interbedded sandstones, siltstones, and mudstones. These rocks are believed to have been deposited as sediments in a fluvial or estuarine environment, and a broad correlation can be made across the Strait. However, at a small scale, the rock layers are not very consistent from pier to pier, and each pier location must be fully investigated and evaluated on its own. Thick layers of sandstone are interbedded with layers of relatively soft mudstone varying in thickness from 5 to 500 mm, most being 50 mm thick. Sandstone is competent rock, but mudstone layers underlying sandstone constitute weak planes for transmitting horizontal forces, such as those from ice and wind. Because of the uncertainty in assessment of the real geometry of these layers, it is assumed that they are present over 100 percent of the foundation areas.

The contractor chose to use spread footings for all the prefabricated piers. For the reasons explained above, the founding level must be on sandstone with the next layer of soft mudstone, if any, following at a depth not less than that determined by the geotechnical analysis on a pier-by-pier basis, but in no case less than 1.50 m. The dredging operations consist of first removing the overburden, which is up to 10 m thick, and excavating a trench to the competent sandstone level; a template is used to guide the dredging bucket in the circular pattern.

The prefabricated pier base ring footing is installed in the trench on three hard points about 0.5 m above the bottom. The three points determine a horizontal level on which to set the pier base and leave a space between the ring footing and the trench, which is then filled with a specially formulated tremie concrete, ensuring uniform bearing of the whole pier base on the rock.

Safety against sliding of the foundation is checked at the interface with the rock assuming a shear friction corresponding to 16 degress/18 degrees for the undrained mudstone. The compressive stresses at ultimate limit state are in the range of 1.2 to 1.6 MPa; actual strength of the rock is twice that value or larger, depending upon the pier location.

The modulus of subgrade reaction is in the magnitude of 110 MN/m^3; long-term settlements are minimal. At the ultimate limit state, the eccentricity of the resulting force applied to the footing is limited to 0.33 of its diameter.

2.2.2 Pier Bases and Pier Shafts

All pier bases and pier shafts were prefabricated in the casting yard at Borden. They were prefabricated separately because of height and capacity limitations of the catamaran-type floating heavy-lift equipment, called Svanen, which was upgraded after previous use in Denmark on the Great Belt Project.

There are two types of pier bases: Type B1 for depths down to -27.0 m; and Type B3 for --27.0 to -38.2 m. B1 has a ring footing, 22.0 m in diameter and 4 m wide, that fits exactly between the two hulls of Svanen. B3 has a ring footing 28.0 m in diameter, but with two flat surfaces spaced 22.0 m so that it also fits into Svanen. Above the ring footing, a conical shell transfers the loads from the barrel, which varies in height according to the depth of the foundation. The barrel ends at elevation -4.0 m by a male cone used to connect the pier shaft to the pier base. The maximum weights of B1 and B3 bases are 35 and 52 MN, respectively. The maximum height of B3 is 42.0 m.

Each pier shaft comprises the shaft itself and the ice shield. It is one of the most critical components of the whole structure because it will be in direct contact with the most aggressive and corrosive environment, sea water in the tidal range, salt-laden spray and air, and abrasion from the ice; therefore all exposed cast-in-place joints were eliminated from this element, which is monolithic from the bottom of the ice shield to the top of the pier, with a maximum weight of 40 MN.

The ice shield is conical with a base diameter of 20.0 m, a height of 8.0 m and a 52-degree angle on the horizontal. It is solid except for a central conical void that matches the top of the pier base. The ice shield itself extends between elevations -4.0 and +4.0 m; it is clad with a 10-mm mild steel sheet for abrasion protection.

The pier shaft has a box section, varying from an octagon at the top of the ice shield to a rectangle at the top of the shaft. The walls are 600 mm thick.

The pier shaft is assembled onto the pier base by being lowered until it rests on hydraulic jacks on top of the base; the position of the top of the pier shaft is adjusted by activating those jacks. The space left between the two cones is grouted, creating a continuous structure through the keyed joint, and vertical post-tensioning tendons crossing the joint are then stressed.

The top of the pier shaft is equipped with a template that is matchcast to the soffit of the pier segment. Once the pier shaft is in place, the template (1.0 MN) is grouted in positi6n so that the future cantilever girder (78 MN) can be placed directly in its final position, thereby avoiding delicate and time-consuming adjustments of a heavy and unstable cantilever.

2.2.3 Superstructure

The superstructure forms a series of frames connected by 60.0-m-long drop-in expansion spans. A frame consists of two cantilevers, integral with the piers and made continuous by inserting a 52.0-m drop-in span between the cantilever tips, pouring closure joints, and stressing post-tensioning tendons to achieve a fully monolithic frame.

3. DESIGN REQUIREMENTS

3.1 Load Combinations and Load Factors

Public Works Canada requires that load combinations and load and resistance factors for ultimate and serviceability limit states be derived specifically for the Project through a full calibration process using probabilistic reliability techniques.

A target safety index β=4.0, applies to each multi-load-path component of the bridge at ultimate limit states, for a 100-year design life. For single-load-path components β=4.25 is imposed. The target safety index is a measure of the accepted risk of failure of a structural member.

As a result of the analysis, load factors, different from those usually recommended in codes, were obtained.

For serviceability limit states, crack widths are related to the change of stress level in the reinforcement or tendon for a given spacing.

3.2 Ice Loading

Generally, the ice season in the Northumberland Strait begins in December or early January, and conditions worsen until late March. The maximum thickness of ice floes, i.e. floating pressure ridges formed from large sheets at the surface of the sea, is about 0.30 m. Floes may occasionally extend over 500 m with a mean of 118 m.

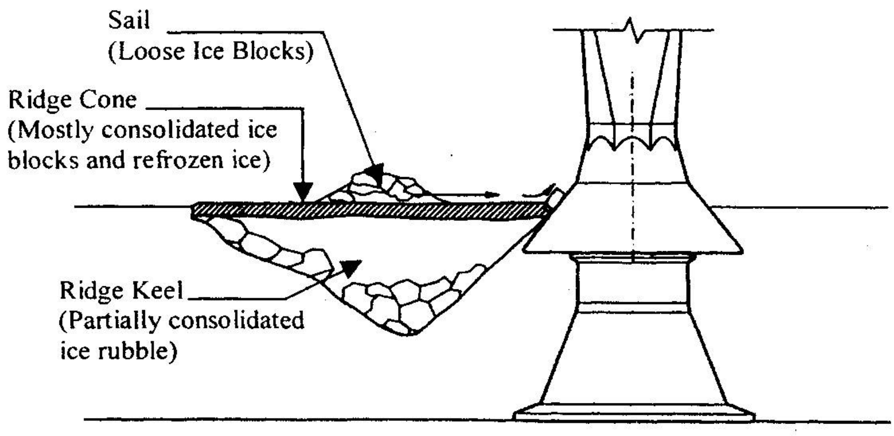

Figure 1: Ice Formations

Currents, waves, and wind induce ice movements, cause floes to break and result in rafting and ridging; ice ridges consist of a consolidated core of refrozen ice at the waterline with loosely bonded blocks of ice forming a small sail on top of the ridge core and a much larger keel below it.

Ridge dimensions can be 50 to 75 m. The ridge keel depths can be evaluated from the number and location of scours seen on the bottom of the Strait; the deepest is at 18 m with an average of 8.5 m. The ridge core thickness may reach 2.5 m.

The critical case for the substructure of the bridge is the consolidated ridge core hitting the pier shaft. To minimize the horizontal force on the pier shaft, a conical ice shield was designed with an angle of 52 degrees to break the ridge core in bending the ice riding up the cone and collapsing under its own weight, rather than crushing directly on a vertical surface, producing a much higher force. The ice shield is clad with ultra high-performance concrete to minimize ice abrasion and reduce friction.

It should be noted that all assessment of ice loads carries a degree of uncertainty as relevant on-site measurements of ice forces are scarce; besides those made at Lighthouse KEMI-1 in the Gulf of Bothnia 1985-1986, practically none have been reported, and laboratory test results can be only a guide.

Another aspect of ice loading is its dynamics. The dynamic response of the bridge allows the assessment of the dynamic loading characteristics of the ice. An analysis was carried out taking into account ice force versus time histories derived from the consideration of all contributing features: ice failure frequencies; ice speed; ridge core characteristics, ridge keel dynamics; rubble surcharge.

Failure of the ridge core is estimated to be the most likely source of dynamic ice loads for frequencies of less than 1.5 Hz; ridge keel dynamics activate frequencies below 1.0 Hz; the combination of ridge core and ridge keel effects is relevant only for frequencies less than 1.0 Hz. The lowest natural frequency considered for the completed bridge is 2.6 Hz, higher than the ice-induced frequencies.

The design of the substructure was based on a transverse static ice force (perpendicular to the bridge) of 17 MN and a longitudinal static ice force (in the direction of the bridge) of 12 MN. For ultimate conditions a factor of 1.8 is applied to the loads, which control the dimensions of the foundations.

4. CONCLUSIONS

The Confederation Bridge is one of the longest deep-water structure in the world. It was designed for 100-years service life in a harsh environment. Unique precasting methods were used for the substructure and superstructure to allow construction of the bridge within a tight schedule. The bridge substructure was designed to resist ice loads up to 30 MN. This unique structure was opened to traffic on schedule in June 1997.

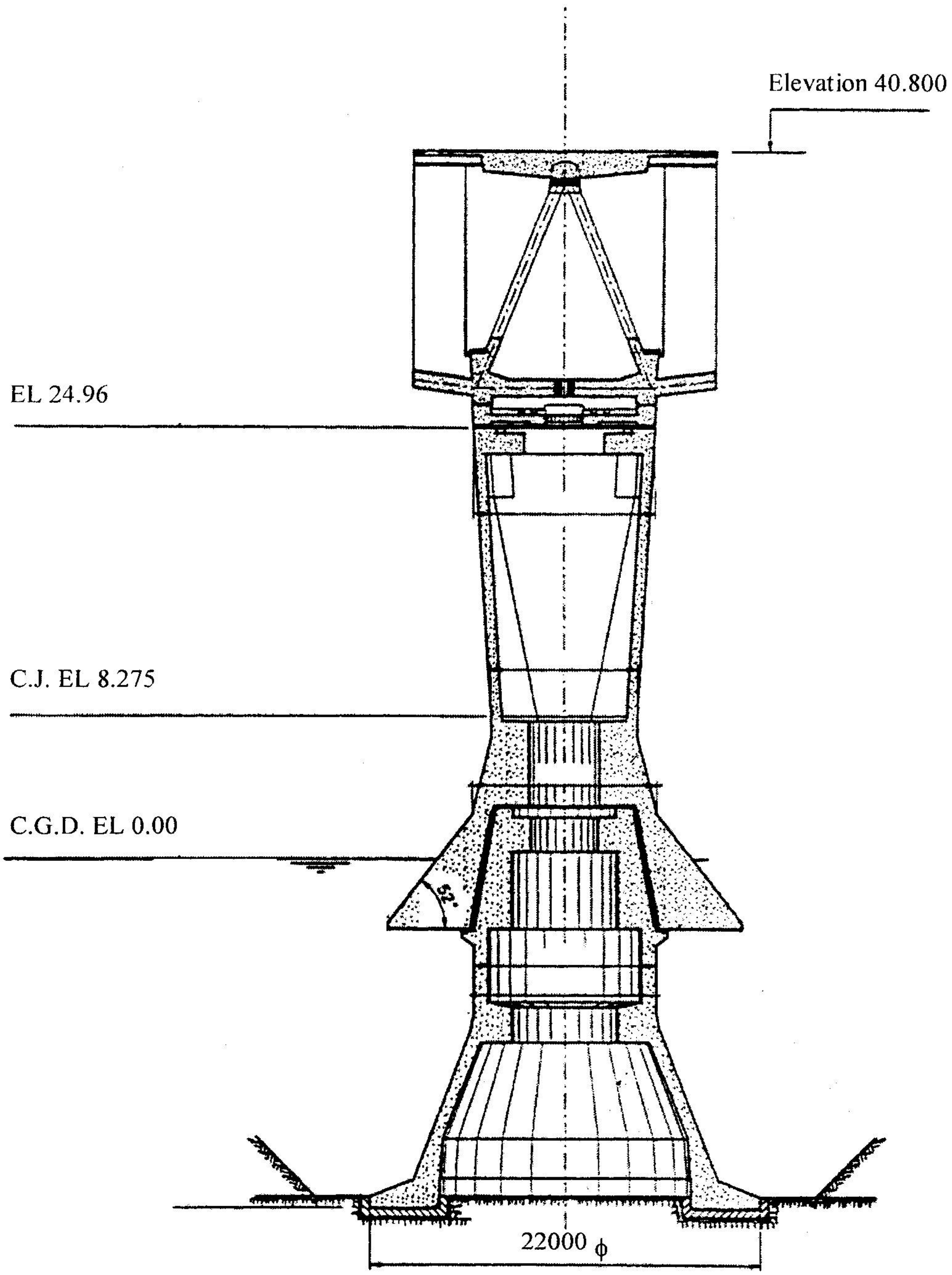

Figure 2: Longitudinal Section