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Session - 4

Loading, Load Factors and Design Techniques (Part - 2)

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SWING BRIDGE OVER SUEZ CANAL AT EL FERDAN SOIL - STRUCTURE INTERACTION AND DEFORMATIONS

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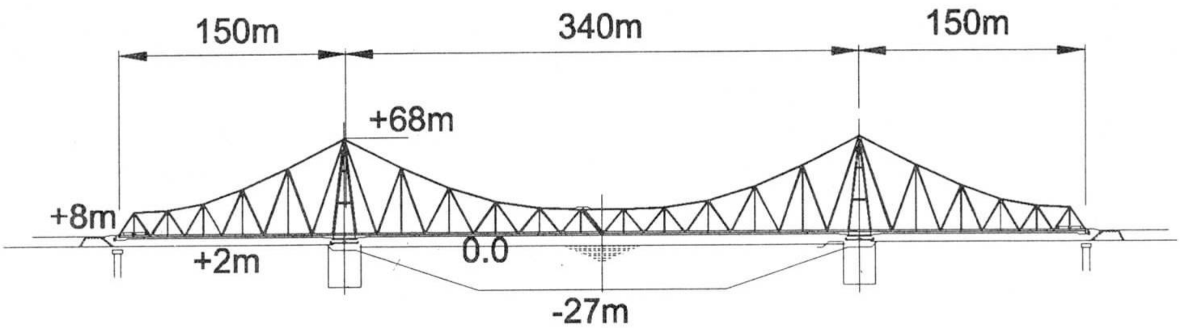
SUMMARY



For the third time a swing bridge over the Suez Canal at El Ferdan (Egypt) is under construction. After completion it will be the largest moveable bridge in the whole world, and this bridge will reinstate the former road and rail link across the Suez Canal.
Taking into account the fact that foundations are in the slope of the Suez Canal (27,5 m deep) and that heavy wind (250 km/h) and a considerable earthquake with an acceleration =150 cm/sec^2 has to be considered, a solution with a steel superstructure and piled piers was proposed by the Consortium KRUPP – BESIX – ORASCOM for the design and construct tender organised by the Egyptian National Railways. Ship collision on the piled piers was also considered.
In the parked position parallel to the Canal bank each of the superstructure halves can be seen as a 150-m single-span girder having a cantilever of 170 m. In the closed position the main span length over the Suez Canal is 340 m both end spans are 150 m.
The foundations consist of a pile-raft foundation composed of 36 bored piles diameter 1,50 m and a rigid pilecap with a thickness of 4,5m. Soil structure interaction is considered for static as well as for dynamic loads.
Design and construction are governed by a quality plan according to the ISO 9000 standard.

The following subjects are developed in this paper:
1 Ship collision
2 Transfer of horizontal loads from superstructure to pile cap
3 Earthquake behaviour of the pile foundation
4 Soil – structure interaction
5 Comparison of calculation models.

1 SHIP COLLISION

According to the specifications, a protection jetty has to be constructed on both sides of the canal in order to protect the bridge in parked position against impacts of vessels. Vessels with a water displacement up to 300,000 ton and a sailing speed of 8 km/h have to be considered.

For the evaluation of the impact of a vessel against the pier, we considered the same several conditions.

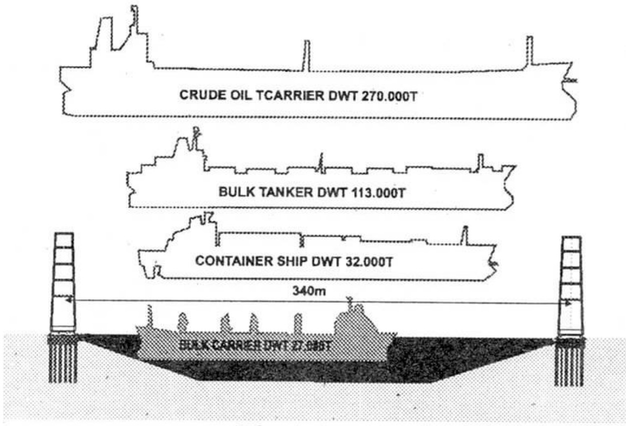


Fig.1.1

Vessels normally sail in the direction of the axis of the canal. If the vessel is out of control, it can deviate from this direction. In that case, the vessel will bump against the slope of the canal.

As long as the collision angle is less than 45° for tankers (cylinder bow) or 26° for container ships (sharp bow), the vessel will slide off the slope. At greater collision angles, the vessel will run up and if the slope is steep dig into the canal slope.

On figure 1.1 one can see that such large vessels cannot sail to the pier in a direction perpendicular to the canal axis. However, this extreme case was analysed to evaluate the impact forces on the pier. In figure 1.2 one can see the calculation method which was applied.

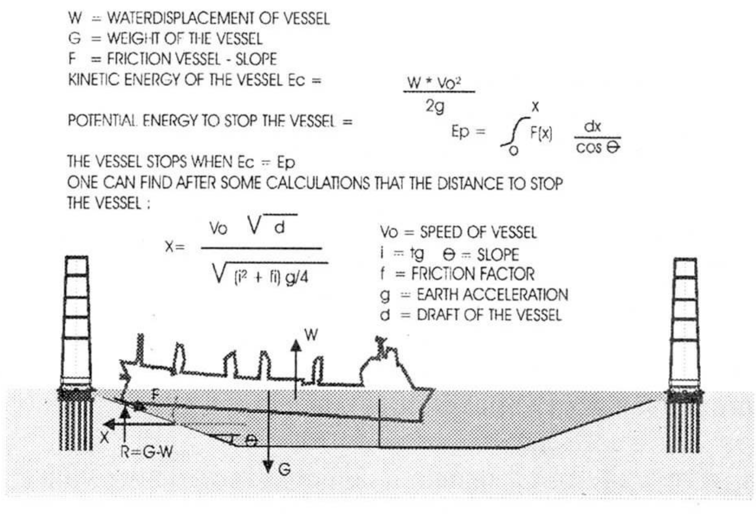


Fig 1.2

The "friction factor" for a vessel which forefoot slides on the slope is 0,4 (ref. 1). This "sliding" assumption is on the safe side, since "digging" generate passive earthpressure and thus also a larger friction factor.

Applying the general formula to the Suez Canal (f = 0,4; i = 1/3), and taking into account that the slope of the canal bank reaches the pier at water level we easily find:

$$\boxed{a = 3d - 1{,}29\, Vo\sqrt{d}}$$

With: a = distance to the pier the vessel stops (m)
Vo = speed of the vessel at the moment that the vessel touches the slope (m/sec)
d = draught of the vessel (m)

In figure 1.3 one can see that only vessels with a draught smaller than 3m can reach the pier.

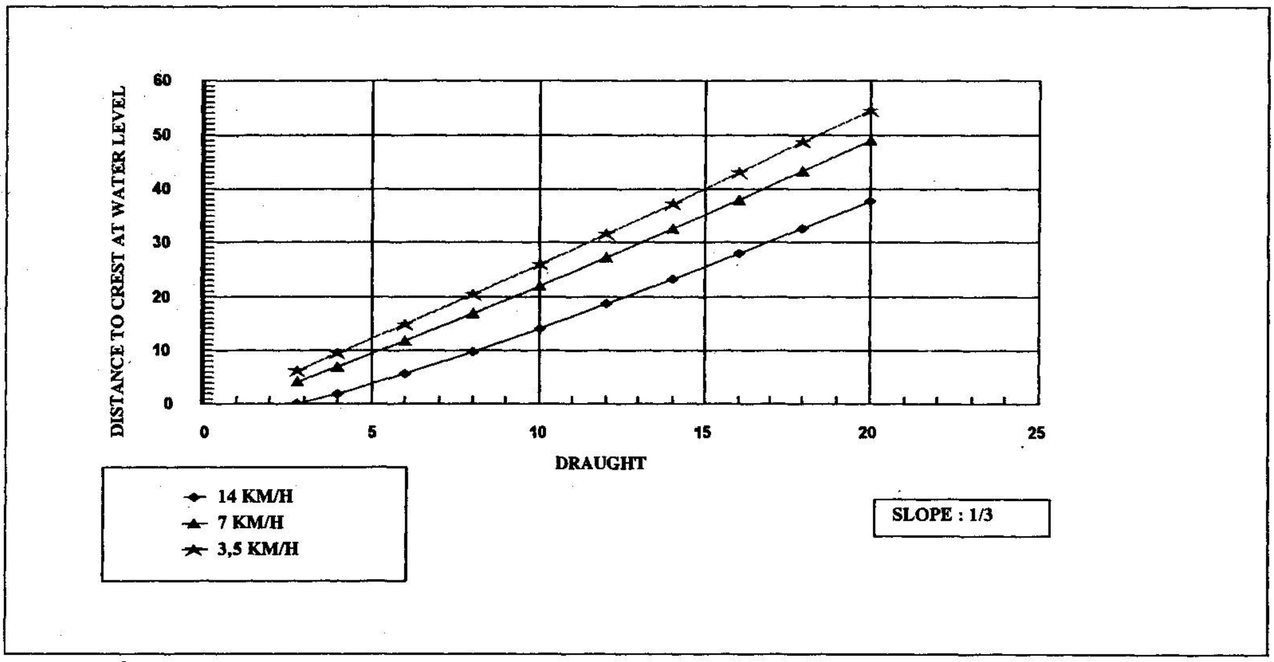


Fig. 1.3

On figure 1.4 and 1.5 (ref. 2) one can see that the impact load is 15 MN (for a speed of 14 km/h = 3,9 m/sec).

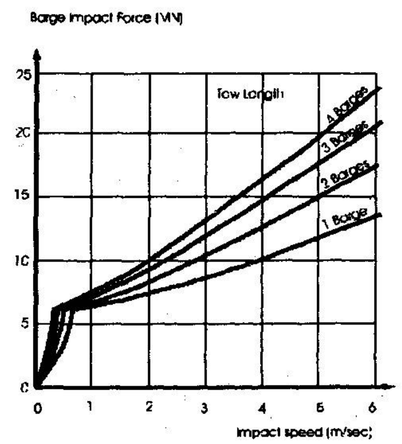


Fig. 1.4

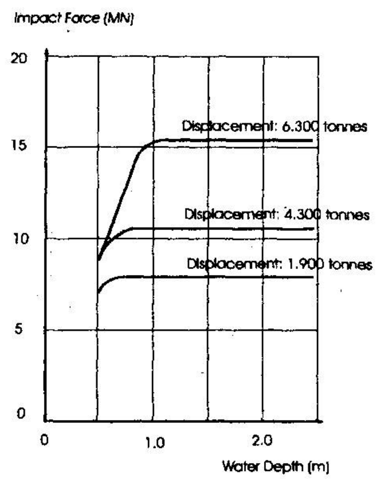


Fig. 1.5

2 TRANSFER OF HORIZONTAL LOADS FROM SUPERSTRUCTURE TO PILE CAP

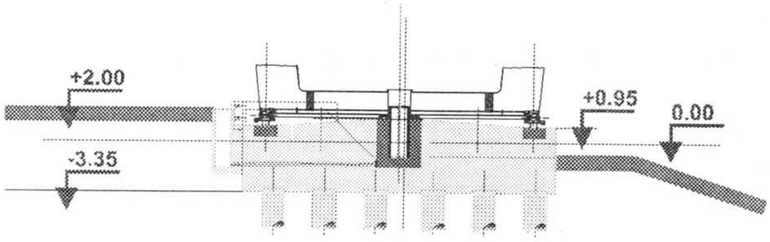


Fig. 2.1

Vertical loads are transferred from the superstructure to the pier cap by mean of a swing gear which consist of a circular structure supported by two layers of conical rolls. In the centre of the swing gear there is a pin to transfer the horizontal loads to the pile cap (Fig. 2.1). The pin itself is a steel cylinder with a diameter of 1,3 m and a wall thickness of 10 cm. The length is 3,5 m and is embedded in the concrete over 2,53 m. The horizontal load to be transferred is 9140 KN under wind load and 16.955 KN under seismic load.

Designers of the superstructure performed the predesign based on a former publication (ref. 3). For the final design more detailed calculations were performed by the designers of the substructure: a 3-D elastic-plastic model was considered (Fig. 2.2).

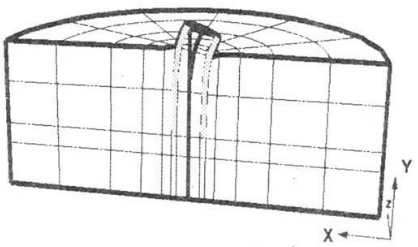


Fig.2.2

The steel tube is modelled as an elastic material and for the concrete we selected a Mohr-Coulomb model with $\varphi = 30°$ and $c = 15.000$ KN/m², which corresponds to an unconfined strength of 30 MPa

Two calculations were performed:
For an horizontal load of 9140 KN, a displacement of 0,66 mm is found assuming that concrete can resist to tensile stresses and 2,9 mm assuming that concrete cannot resist to tensile stresses.

To define the reinforcement around the pin, horizontal planes were calculated: in figure 2.3 one can see the deformations and the tensile stresses in the concrete assuming tensile can occur between pin and concrete, in figure 2.4 no tensile is allowed between pin and concrete, one can see a gap between concrete and pin. The reinforcements are calculated using the isolines of tensile stresses given in figure 2.4.

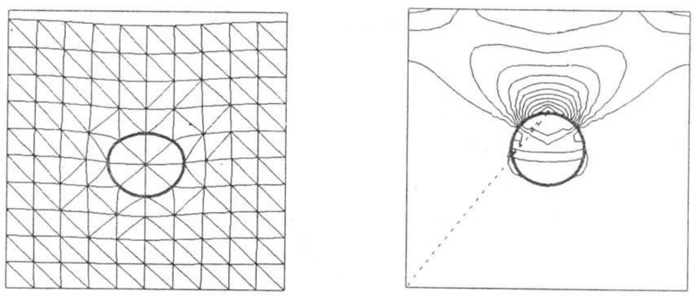


Fig. 2.3

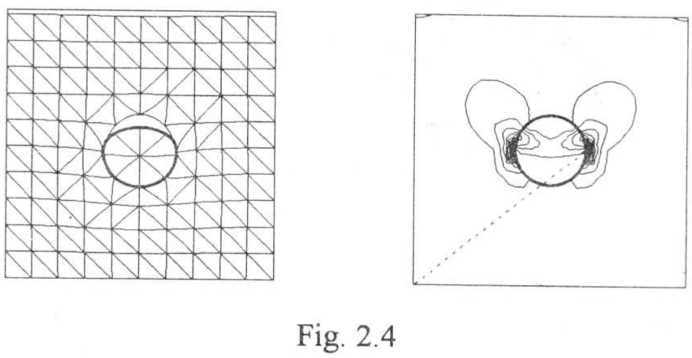


Fig. 2.4

3 EARTHQUAKE BEHAVIOUR OF THE PILE FOUNDATION

Earthquake calculations were performed according to different methods:
5.1 By the Consortium
5.1.1 For the superstructure using Response Model Analyse in SAPN based on the Acceleration Response Spectrum calculated by Dr. O, Ramadan.
5.1.2 For the substructure using DYNA IV (Novak) which is a 3-D Dynamic elasto-plastic model.

5.2 By the Consultant
The consultant performed own calculation.
It is interesting to evaluate the dynamic calculations by comparing the “earthquake coefficient” this is the factor you have to apply to the vertical loads to find the horizontal loads for pseudo dynamic calculations (for a ground acceleration of 150 cm/sec^2).

Location	Superstructure		Pile Foundation	
	Consortium	Consultant	Consortium	Consultant
Horizontal load (KN)	16.128	11.868	16.995	21.600
Vertical load (KN)	75.890	72.667	129.147	129.147
H/V	0,20	0,16	0,13	0,17

4 SOIL-STRUCTURE INTERACTION

For a major bridge more particularly when deformations are very important, one cannot assume that the superstructure, the pile cap and the piles behave independently i.e. that the stiffness of this structural elements together with the stiffness of the soil do not influence each other.
More advanced computer programs allow modelling the pile cap and piles as elastic elements together with the soil as elastic-plastic elements linked together by interface elements.
For the El Ferdan Bridge, various calculation methods were applied to evaluate the deformations of the foundations and the bending moments in the piles.
Intensive soil investigations including borings, SPT, triaxial tests, oedometer tests and PMT were carried out. The selection of the soil parameters for the calculations is very important and the input of experts is required for major structures.

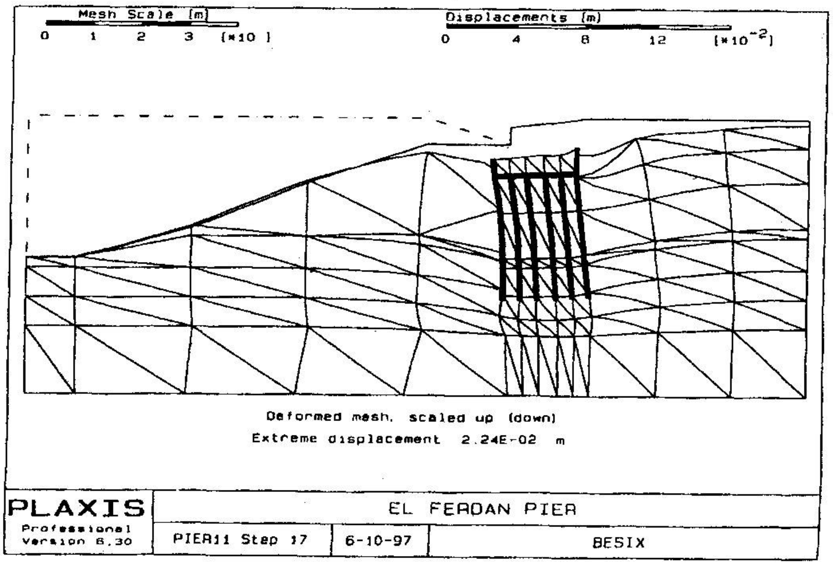


Fig. 4.1 Open Bridge Hmax – Deformations

For the dense sand layers ($D_r > 80\%$), we assume $\alpha = 32°$ and $G = 20.000 + 800\ z$ (KN/m²), with z = depth in meter. For the clay layers, ($I_p = 50$, $C_u = 250$ KPa) we assume $\varphi = 25°$ $C = 25$ KN/m² and $G = 12.000$ KN/m². The G-values are G_{50} values for long term behaviour. The interaction factor (= $\tan \alpha_{pile} / \tan \varphi_{soil}$) is 0,7 for sand layers and 0,6 for clay layers.
The history of the stresses in the soil is repeated in the different steps of the Plaxis FEM calculations. Starting from the initial situation (before dredging of the canal), initial stresses are generated. Then following steps are analysed: Dredging of canal, installation of piles, pouring of pilecap, erection of bridge, and other loadcases (see Fig. 4.1). The results concerning the deformations are given hereafter:

Case	**Horizontal displacements (mm)**	**Vertical displacements (mm)**	**Rotation (rad x 1000)**
1. Dead load bridges	1,45	15,61	-0,53
2. V max	5,94	21,77	-0,24
3. M max	7,64	20,95	0,39
4. H max (closed bridge)	4,75	15,53	-0,34
5. H max + V max (closed bridge)	6,95	21,01	0,07
6. H max (open bridge)	8,33	16,67	0,15
7. Earthquake	17,27	15,80	0,68

5 COMPARISON OF CALCULATION MODELS

Design is not only compute, it is also evaluate and decide. As an example, the values of Maximal Bending Moments in the piles are given according to different calculation models:

MAXIMAL BENDING MOMENT IN PILES (Serviceability state)			
Horizontal load	Type of calculation	Maximal moment (KNm)	
		H (V = 0)	H + V
Wind	Handcalculations according Franke	607	-
Wind	Simplified model	2.114	-
Wind	Plaxis calculations	-	3.318
Wind	M-Pile calculations	-	4.363
Earthquake	Handcalculations according French standard	705	-
Earthquake	Plaxis calculations (pseudo-dynamic)	3.354	5.874
Earthquake	DYNA IV (full 3 D – dynamic)	3.516	-
Earthquake	M-Pile (pseudo-dynamic)	-	7.140

Finally a bending moment of 8707 KNm was considered for ultimate design state.

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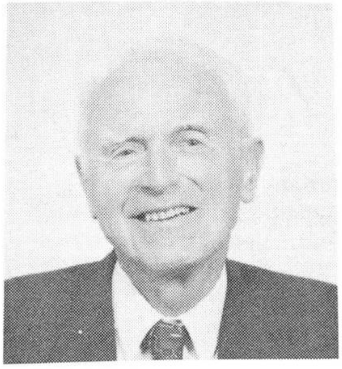
Large Diameter Steel Tubular Piles for Optimum Seismic Performance



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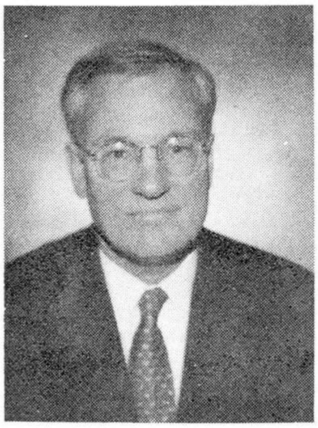
Ben C. Gerwick, Jr. has also served as consultant on the foundation of major overwater bridges worldwide.



Robert Bittner
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Summary

Large diameter steel tubular piles, installed vertically or on a slight rake, offer many advantages as the foundations of major overwater bridge in regions of high seismicity. By proper selection of diameter and wall thickness, an optimum balance between stiffness and strength can be achieved. Under strong seismic motion, the piles will deflect and bend so as to reduce the acceleration forces transmitted to the superstructure, while still limiting drift.

A major advantage of this concept is that the mass of the footing block is significantly reduced from that of the conventional pier. The design of the footing block requires great care in order to transfer the high shears and moments.

Where the soils consist of significant depths of sediments, the kinematic interaction of soils, piles and structures must be considered. Moments will be a maximum just under the footing block. Filling the top portion of the pile with concrete will prevent local buckling and assure ductile behavior even under overload.

Typically the piles will be long, with most or all of their support furnished by friction and will require the use of very large pile hammers to achieve the required capacities. Where the piles are founded on rock or in hardpan, sockets will be drilled and filled with reinforced concrete. For these piles, high moments can occur both at the head and just below the rock surface, so the concrete infill should extend the full length.

The paper will describe the use of tubular piles driven through the deep silty sands of the Jamuna River, Bangladesh, and also where socketed into the near-surface rock for the main pylon pier of the new San Francisco-Oakland East Bay Bridge in California.

Introduction

1.1 Steel tubular cylinder piles with diameters of one to two meters have been previously employed for a number of major bridges, including the Rio Niteroi Bridge in Brazil, the Parana Bridges in Argentina, and on the new bridge across the Tagus River in Portugal. They have proved to be an efficient and economical solution for the foundations, because they enabled the footing block to be constructed near the waterline, thus obviating the need for a deep cofferdam or caisson.

In recent years it has been recognized that tubular piles of even greater diameter, 2.5 to 4.0 meters, have the ability to provide lateral stiffness to the pier along with ductility for overload events such as earthquakes, sea ice, and ship collision. (Fig.1) Combined with an efficiently designed footing block that develops both moment and shear, they can be optimized for drift (lateral deflection), and strength.

These piles are typically thick-walled so as to enable effective driving to deep penetrations, as well as to provide moment resistance and stiffness. The wall thickness can be varied along the length to suit the demands of both driveability and in-service stresses. Where piles are driven into deep sediments, the highest bending moment occurs in the 1 to 1 ½ diameters just below the footing block. Filling the upper part of the pile with concrete, locked to the steel pile by means of shear rings, will give the pile increased stiffness and ductility. Stable hysteresis responses have been obtained in laboratory tests up to a ductility factor of 4.

As a result of the failure of many raked piles in wharves during earthquakes, the current trend has been to use only vertical piles in seismic zones. Vertical piles respond well to excitation from all directions. However, it has been recently realized that in soft sedimentary soils or deep water, a slight rake can be utilized and still not result in excessive moment, shear or axial forces during earthquake. The benefits are greater lateral stiffness under serviceability environmental loadings, reduced drift during the seismic event and especially reduced residual displacement.

1.2 Design of these piles for lateral forces such as those of earthquake, requires a full analysis of the soil-structure interaction. The free-field motions are modified by the kinematic interaction of the pile with the soil due to the pile's stiffness. In layered sediments, the load-deflection performance can be determined by the finite element method with soil springs input along the pile length.

Group effects due to "shadowing" must be considered. For extreme deflections, the moments and deformations resulting from eccentric loading may be important.

1.3 Installation of these large piles requires the use of very large crane barges and very heavy pile hammers.

Templates are usually employed to position the piles accurately, especially when the piles are battered (raked). See Fig. 2. Because there will be only a few such piles in a footing, each will be highly loaded e.g. to several thousand tons. Tolerances in pile head location are very critical.

1.4 Design of the footing blocks requires consideration of the demands for moment, shear and axial force transfer and also the practicality of construction. Precast concrete shells are often employed, temporarily supported on the piles. Both circumferential and axial shear keys are employed. Because of the high forces in moment and shear, post-tensioning may also be required.

It should be noted that if the piles are slightly battered, the footing block can be reduced in size, thus reducing the mass which is subject to high accelerations under earthquake. It has been found that the largest contributor to the shear under earthquake is usually the mass of the footing block. Piles are usually designed for essentially elastic behavior under the design earthquake, with ductility employed for possible overload during a few cycles.

1.5 Large diameter steel tubular piles were utilized for the recently completed Jamuna River Multipurpose Bridge in Bangladesh, where very deep and unstable sediments, combined with variable river depths due to shifting channels as well as high Monsoon floods, demanded an unique solution. The piles varied from 2.5 to 3.15m in diameter. They were 80m in length, with wall thicknesses up to 65mm. They were driven in three sections, using a Menck hydraulic hammer developing 1800 KN-m of energy per blow. The piles were driven through medium dense to very dense sand, and founded in a gravel stratum at a depth of 80m below normal water level. Up to 12,000 blows per pile were required. Splicing of three sections was by full-penetration welding.

The 300 ton pile hammer and the pile sections were handled by a large offshore-type crane barge. Piles were installed at a rate up to 5 per week and the work was completed on schedule.

1.6 Tubular piles are also an excellent solution for overwater bridge piers founded in rock or hardpan. In this case, vertical piles will usually prove best. They are cleaned out and a socket drilled into the rock.

Past practice has then been to install a reinforcing steel cage and to fill the pile with concrete. However, if there is inadequate overburden, the pile may develop high moments at or just below the rock line. In such cases, an oversize socket is drilled and the tubular pile lowered or driven into it, assuring that full moment capacity is provided at this critical region. The annular space is then grouted and the pile filled with concrete.

This concept has been adopted for the seismic retrofit of the several major bridges across San Francisco Bay, in order to develop greater stiffness and thus reduce drift deflection to acceptable values. Steel tubular piles, socketed into rock, have been selected as the probable design for the major pylon pier suspension span of the new San Francisco-Oakland East Bay Bridge while driven tubular piles, 2.5m diameter and up to 100m in length will support the long approach viaduct.

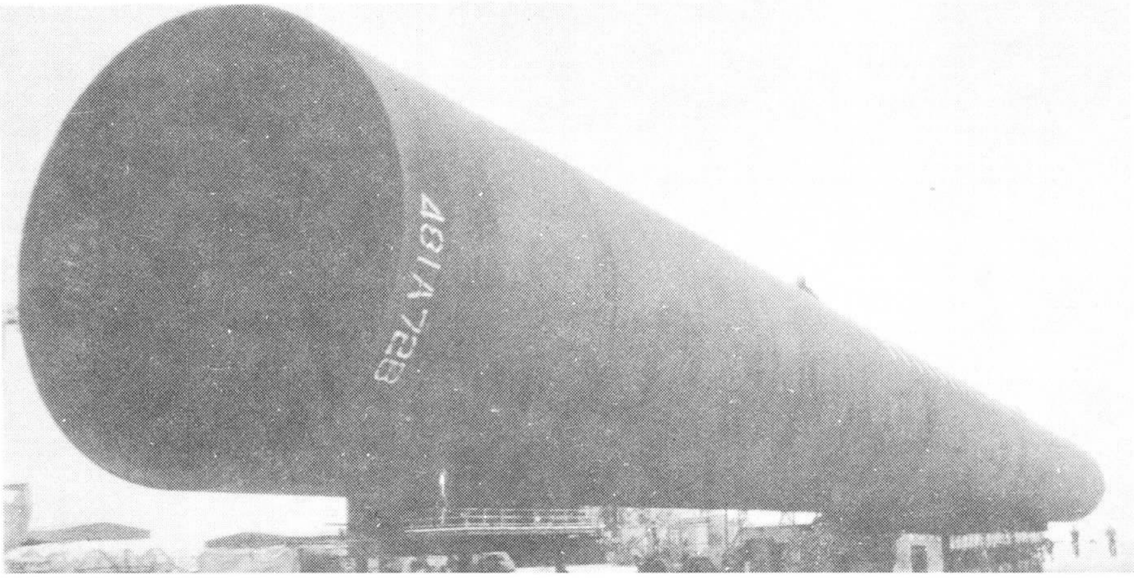

Fig. 1 – 3m dia x 80m long piles were used to support the piers of the Jamuna River Multipurpose bridge in Bangladesh

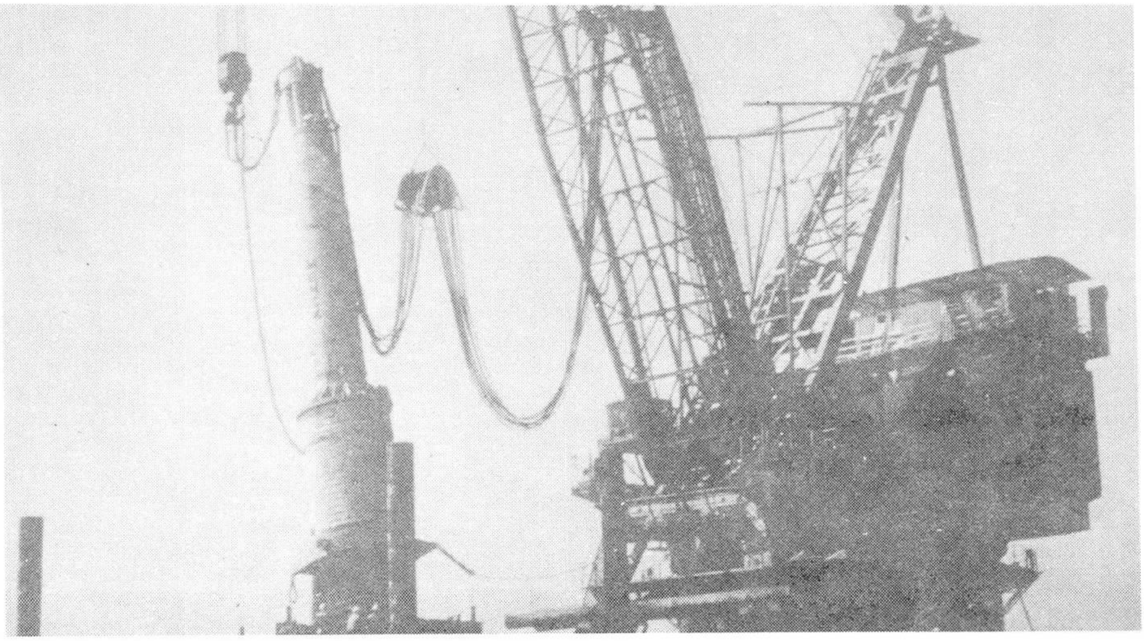

Fig. 2 - Installation of these large piles requires very heavy pile hammers

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LATERAL LOAD CAPACITY ESTIMATION OF LARGE DIAMETER BORED PILES AND ITS IMPLEMENTATION - A STUDY

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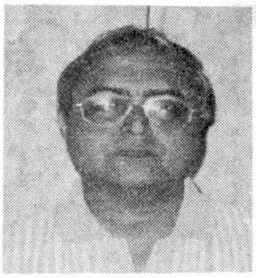

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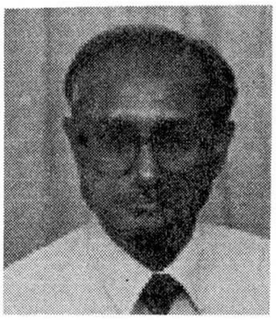

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Summary

Bridge foundation, using large diameter bored piles are presently very common in India due to need of faster implementation, environmental hazard mitigations and also availability of modern equipments. Long span and continuous superstructures transmit large horizontal forces to the substructure and foundation system, under serviceability as well as accidental/occasional loading conditions. Adopting modern seismic dampners and transmission units one can limit or minimise to certain extent the transfer of lateral load on the foundation system. Even after the dampening, the residual horizontal force is often quite high, which needs to be resisted by the foundation system. Proper assessment of lateral capacity of piles is thus important to cater for horizontal forces, particularly in the design stage, to work out appropriate foundation for the structure.

Designed capacity of piles is dependent on many factors, some of which can be determined with reasonable accuracy and the assessment of others is difficult. Assessment of soil properties, which contributes largely towards the lateral load capacity of the piles, is quite

difficult. Further, the lateral load capacity of a group of piles depends on the permissible lateral movement of the pile at the pile cap level satisfying the serviceability requirements and design tolerances for the proposed structure. The embedded length of pile vis-à-vis its sectional properties plays a major role in deciding the behavioural pattern under lateral load. Long term behaviour of pile material and soil, the loading patterns on structure, scour of river, etc. also play important role in the lateral load carrying capacity of a pile. This paper discusses all these aspects.

Often lateral load capacity of a pile is decided based on the load test conducted on individual piles. The authors feel that deciding lateral load capacity for the design of a pile foundation, only on the basis of load tests, which indicates the capacity at the time of testing and very often do not depict the actual long term situation, is not desirable. A single pile which moves laterally during load testing does not provide a direct answer to the various other aspects controlling/affecting the lateral resistance both for single pile or a pile group.

The paper discusses the authors' approach to determine lateral load capacity in compatible with the provision of Indian Standards and applied in their recent projects in India.

1.0 INTRODUCTION

Use of bridge foundations with large diameter piles, whether in land or in rivers with low scour has become quite common in India. Bridge structure is subjected to reversible longitudinal & transverse horizontal forces, and, raker piles, commonly used till eighties, were adopted to take care of these lateral forces. Gradual change from use of smaller diameter driven piles to high capacity large diameter bored cast-in-situ piles, which are more environmental friendly during installation, and faster in implementation has onset a trend to provide vertical piles for resisting, both, vertical & horizontal loads. Accordingly the assessment of horizontal load capacity of pile in addition to vertical capacity has become an important issue in the design.

The ultimate resistance of a vertical pile to lateral load and the deflection of the pile as the load builds up to its ultimate value are complex matters involving the interaction between a structural element and the soil in which it is embedded. The structural elements as well as the soil deform partially elastically and partially plastically. Elastic behaviour of the soil is only an idealised assumption. The deflection pattern of a pile depends on its boundary condition particularly at top where it may be free to rotate or rigidly fixed to a pile cap or is free to rotate at top of soil level but is fixed at much higher level in a pile cap such as foundation in a scoured river.

In majority of cases in India, piles are embedded in soil and are non-socketed. The lateral capacity of these piles depends to a larger extent on the properties of soil. Piles with tips socketed in rock however, have a different approach wherein the socketed area and/or structural capacity of the pile governs the lateral capacity.

Till recent years, due to the very complexity involved in the determination of lateral load carrying capacity, the capacity of piles were being assumed as an adhoc percentage of the vertical load carrying capacity of piles without going in for any analytical estimate. Raker piles were extensively used to cater for high lateral loads. As explained by Chakrabarti A. design of foundation with raker piles in combination with vertical pile could be solution, when a major portion of the horizontal load acts in reversible manner if the lateral load capacity of a pile is considered as an adhoc percentage of its vertical load. Further, installation of bored piles in a rake, calls for use of liner for the full length making it more difficult to implement.

2.0 FAILURE MECHANISM

2.1 ***Soil Behaviour :*** The soil closer to the top surface initially resists the lateral load on a vertical pile. At comparatively lower loads, the soil behaves elastically and also transfers pressure from the top to the deeper stratum. With increase in load the topsoil tends to become plastic and transfer more load to the lower depths. The behaviour of soil reaction again depends on the width to depth ratio of the pile and its restraint conditions at the top level.

2.2 ***Pile Movement/Rotation :*** A short rigid pile, unrestrained at the top with length/diameter ratio upto around 12, tends to rotate about a point slightly above the bottom tip and passive resistance develops in the portion below the point of rotation whereas the top is subjected to active pressure. However, the same pile with a rigid pilecap tends to deflect/shift laterally en mass and active resistance develops for the entire length.

The failure mechanism of a pile with longer length of embedment is differenent. The passive resistance of the lower part of the pile is infinite, thus rotation of the pile is difficult occur and it remains vertical. Only the top deflects either with uni-directional rotation like a cantilever for a free pile head and like a propped cantilever or both end fixed condition compatible with level of fixity.

2.3 ***Failure Criteria/Serviceability Criteria :*** The ultimate failure for a long pile occurs when the moment on the pile induces a plastic hinge. But this phenomenon occurs after a large lateral displacement as well as rotation (for free piles heads) have taken place. But in most of the cases such large movements of the foundation system creates serviceability problems for the bearing system as well as the superstructure. Thus the lateral load capacity of long piles are decided on the basis of limiting lateral deflection at the pile top.

3.0 LATERAL LOAD CAPACITY CALCULATION

3.1 ***Criterion :*** As stated earlier the criteria for deciding the lateral load capacity of a long pile under different soil condition, pile rigidity and fixed end conditions of the pile is the limit upto which lateral deflection of the pile at the ground surface/pilecap level (as the case may be) can be permitted. It is not to be the ultimate lateral load resistance of the pile, considering a total soil structure interaction between a semi rigid structure and non-homogenous, often non-elastic soil mass, before the pile is subjected to its ultimate moment capacity causing formation of a hinge in pile structure in the embedded portion.

3.2 ***Approaches of Analysis :*** Two approaches that have been professed by various authorities, although both of them have limitations, are :

a) The elastic approach, where the soil is assumed as an ideal elastic continum.
b) The sub-grade reaction (winkler model) approach where the continuous nature of the soil medium is ignored and the pile reaction at a point is simply related to the corresponding deflection of the pile at that point.

Elastic Approach : The very important and fundamental aspect of the approach is that under pure elastic condition the horizontal displacement of soil (be it in the direction of applied load or in the opposite face) and the pile are equal. The elastic analysis thus assumes that the soil behind the pile adheres to the pile at all times which is basically not correct. The elastic approach also makes a definite distinction between socketed pile and non-socketed pile.

To consider soil as an elastic continum is more satisfactory assumption as it accounts for the continuous nature of the soil. Perhaps this may be a more appropriate approach for mathematical representation when group action of piles are taken into account. But the near impossible and daunting task is to determine the elastic soil modulus, even for single layer. Whereas for long piles the soil affected is mostly layered. Thus this is not a common approach, though several authors have deliberated on this approach.

Subgrade Reaction Modulus Approach : This is a concept which was even conceived by Terzaghi. The approach characterizes the soil as a series of unconnected linearly elastic springs, so deformation occurs where only loading exist and interalia soil resistance is mobilised when deformation takes place. The obvious disadvantage of the approach is the basic assumption of lack of continuity in all direction whereas in real situation the soil is some what continuous. The elastic spring constant, commonly known as modulus of subgrade reaction, is dependent on the width of pile and often on the depth of the particular loaded area of the soil.

The Most Accepted Approach : The exact solution to the problem of a laterally loaded flexible or semi rigid pile in an elastoplastic soil mass is a complicated and difficult one in a multi dimensional continum mechanism. As such it warrants the designer to adopt certain measures and assumption, which may tend to a trifle adhocism. However considering all factors the method of subgrade modulus reaction is the most acceptable practice as it provides a relatively simple means of analysis and enables factors such as non liniearty, variation of soil stiffness with depth, layering of soil profile etc. to be taken into consideration.

Comparison of displacement and rotation factors for long piles by adopting the elastic method and those determined by subgrade modulus method have shown higher values by the later method. As such it is felt that adopting the subgrade modulus method, even if it gives conservative values, the pile lateral load capacity calculation is on the safer side for the structure.

4.0 CALCULATING RESISTANCE TO LATERAL LOAD

4.1 ***Uncertain factors*** : Although the modulus of subgrade reaction approach has been extensively studied but the research has not yielded any simple design method which can easily and universally adopted to any soil or type of pile. There are many inter-related factors and a few dominant factors include –

a) The pile length and its stiffness alongwith the soil parameters decides the failure mechanism i.e. whether the entire pile will rotate for deflection or the major part of the embedded length will remain vertical.
b) The type of loading, whether sustained or alternating or pulsating or accidental and their influences & the degree of yielding of the soil.
c) Scouring of soil mass around the pile of the pile group, particularly for river bridges in alluvium deposits.

d) Seasonal shrinkage of clayey soil away from the upper part of the pile shaft etc.

4.2 *Need for Simplification :* After detailed deliberations most of the researchers/authors have suggested only simplified procedures as elaborate calculation processes are not justified, because of the non-homogeneity of the natural soil deposits and the disturbance to the soil caused by pile installation. None of these factors, though very significant, can be reproduced in the calculation method. Various simplifications have been necessary in order to provide solutions to this complex problem of soil structure interaction. The Bureau of Indian Standards (BIS) has, after reasonable studies, suggested certain simplified methods as detailed later. The authors have used these simplified methods and guidelines of BIS for load testing and its acceptance criteria to decide the theoretical safe lateral load capacity of vertical piles for permissible deflection.

4.3 *Assessment of Modulus of Sub-grade Reaction :* The modulus can be determined by :

a) Full scale lateral load test on a pile
b) Plate load test
c) Empirical correlation with other soil properties

The method at (a) has been used by several researches e.g. Matlock & Ripperberge, Reese & Cox etc. but they are time consuming, requires care and are very expensive. The method is also prone to give wrong values due to inadequacy in instrumentation.

The pioneers in the field like Terzaglu, Broms etc. have discussed method (b) in detail but ultimately attempts have been made by them to co-relate these results with the other elastic parameter and properties of the soil.

Ultimately most of the researcher in this field has tried to co-relate the basic nature and property of the soil to an empirical value of the subgrade reaction. BIS has specified certain values to be adopted which have been discussed later.

4.4 *Group Effect & Repeated Loading* : It has been often emphasized that for vertical capacity of a pile group, the group action plays a major part. The authors while working on the lateral load capacity of a pile foundation system based on the capacity of a single pile have been often advised that the method adopted is quite conservative as pile in a group will have larger capacity. However Davisson (1970) sates that the spacing of piles in the direction of the load is of high importance. No positive decision in this respect can be taken to have rectangular grids in Indian condition mainly, due to seismic load (which constitutes major part of lateral load) and can act in any direction. There are various theories on the group effect of piles based on loading but the ideal spacing of 8d in the direction of the load as suggested by a few researchers is highly impractical.

Similarly, repetition of loading has its effect on the soil structure interaction and on the lateral load carrying capacity but again for bridge structures the lateral loads, which are the major contributors are neither pulsating nor very frequent.

4.5 *Effect of Consolidation and Creep :* As a result of consolidation and creep of the soil surrounding a laterally loaded pile, considering the elasto plastic nature of soil, an increase in lateral deflection and a re-distribution of soil reaction will occur each time. Though for fully granular and non-cohesive soil the value effective subgrade modulus (k) equals to the test results but with cohesive soil the relation of the former to later is always less than unity.

Due to long term effect of repetitive loading, consolidation and creep, it is desirable not to design a pile foundation on the basis of load test only. These results may only be used as a confirmatory data to the theoretical safe capacity, as adopted in design.

4.6 ***Effect of Scour and Future Construction*** : For the design of pile foundation of a bridge structure it is necessary that the depth of design scour level (2d for piers and 1.27 d for abutments) are determined and the scoured condition is assessed for determination of the capacity of the pile. It is almost impossible to simulate a deep scour condition for a pile load test with facilities for adequate deflection and rotation of a free standing pile upto its top.

Further for flyovers in land where there are possibility to construct an underpass/subway adjacent to foundations in the near future, the pile capacity should take into account such provisions.

5.0 METHOD ADOPTED BY THE AUTHORS

5.1 ***Provisions in BIS Code to Determine the Lateral Deflection:*** The BIS Code stipulates the followings for determination of lateral deflection at the pile head and the depth of fixity.

5.1.1 The long flexible pile, fully or partially embedded, is treated as a cantilever fixed at depth below the ground level (see Fig. 1)

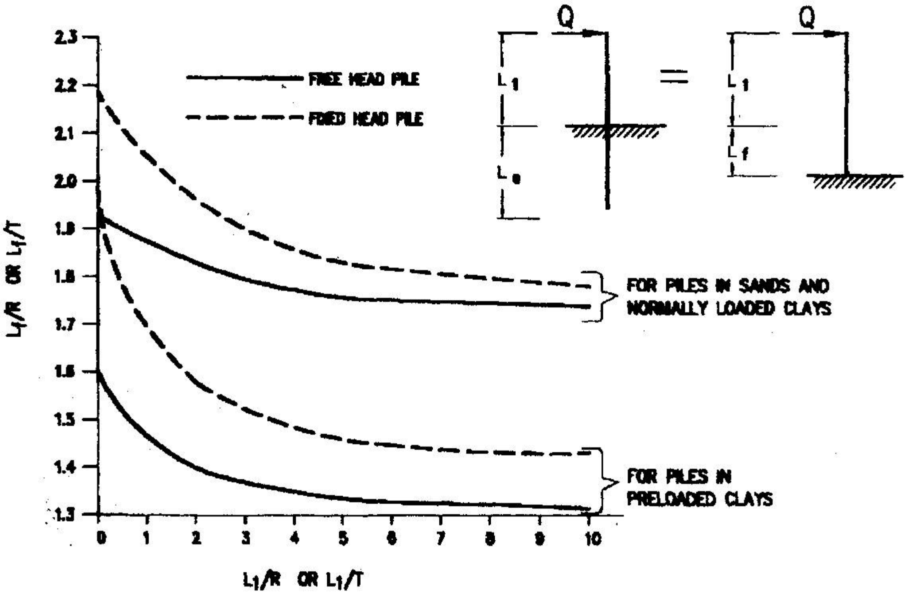

FIG. 1 : DETERMINATION OF DEPTH FIXITY

5.1.2 The depth of fixity and hence the equivalent length of the cantilever are determined using the plots given in Fig. 1

Where $T = 5\sqrt{EI/K1}$ and $R = 4\sqrt{EI/K2}$

(K1 and K2 are constants given in Table 1 and 2 below.
E is the Young's modulus of the pile material in kg/cm^2 and I is the moment of inertia of the pile cross-section in cm^4)

TABLE 1 : VALUES OF CONSTANT K1 (kg/cm^3)

TYPE OF SOIL	VALUE	
	Dry	Submerged
Loose Sand	0.260	0.146
Medium Sand	0.775	0.525
Dense Sand	2.075	1.245
Very loose sand under Repeated loading or Normally loading clays	---	0.040

TABLE 2 : VALUES OF CONSTANT K2 (kg/cm^3)

UNCONFINED COMPRESSIVE STRENGTH IN kg/cm^2	VALUE
0.2 to 0.4	7.75
1 to 2	48.80
2 to 4	97.75
More than 4	195.50

5.1.3 Knowing the length of the equivalent cantilever the pile head deflection (Y) shall be computed using the following equations:

$$Y = \frac{Q(L_1 + L_F)^3}{3\,EI} \text{ for free head pile } = \frac{Q(L_1 + L_F)^3}{12\,EI} \text{ for fixed head pile}$$

where Q is the lateral load

5.2 ***Admissible Deflection to Decide the Safe Lateral Load as per BIS*** **:** The BIS Code for lateral load testing on piles specifies that safe lateral load on the pile shall be taken as the least of the followings displacements to be considered at the cut-off level of the pile :

a) Fifty percent of the final load at which the total displacement increases to 12 mm
b) Final load at which the total displacement corresponds to 5 mm and
c) Load corresponding to any other specified displacement as per performance criteria.

It will thus be seen that the stipulations of the code based on empirical method specifies that the safe lateral load capacity of a pile foundation should determine on the basis of a deflection of 5 mm at the cut off level of the pile for an individual pile.

5.3 ***The Safe Lateral Capacity Determination*** **:** The authors, as such based on the soil bore data and classification as well as the physical properties of the pile has worked out values of L_f and K1/K2 as per para 5.1.2 above. The effects of layered soil have been considered by repeated trial i.e. at the first instance only the value of K1/K2 for the top soil has been taken and when L_f exceeds the depth of the top layer weighted average of K1/K2 has been taken based on the depth of the layers between L_f. After a few trials the L_f considering the weighted average of value of K1 or K2 equals the calculated value. Once the L_1 and L_f are known the safe lateral load for individual pile has been determined as equal to Q which produces a deflection (Y) of 5 mm at the pile head level.

Effect of group action has not been considered due to the complexities involved but the spacings of the piles have been kept at least 3 times the diameter of the pile.

5.4 ***Analysis of Load Test Results*** : The lateral load capacity of individual piles have been calculated by the authors using the above method for some of the projects. Subsequently the pile have been load tested for lateral load after their installations. The sites constituted mainly of non-cohesive soil though top layer has been silty sand or silty clay, with predomination of silt.

It has been observed that the load deflection curve changes its slope drastically near or just beyond the theoretical safe lateral load as calculated above. Lateral deflection observed at the top of the pile at this load hardly reaches 1 mm but after crossing this load the rates of deflection per unit lateral load increases substantially. These results often indicated that a very conservative value of safe lateral load is being considered for the design. But considering the various factors that effects the lateral load capacity in the long term due to repeated loading, group effect, creep of soil etc. the method adopted in deciding the lateral load capacity of the piles for the design appears rational.

6.0 CONCLUSIONS

From the above, it can be concluded that determination of lateral load capacity of pile foundation system is reasonably complex and simplified approach adopting empirical values of soil properties and empirical methods needs to be considered.

It will however, be desirable to decide the vertical design capacity of pile judiciously, even for the layered soil considering the codal provisions of the country.

It is very difficult to make an elastic analysis of pile group subject to lateral load when these also subjected to vertical load. Load testing of piles concurrently with vertical and lateral load is not possible although in actual, the soil surrounding the piles is subjected to both lateral and vertical strains/stresses be it in elastic conditions or plastic condition. As such a little conservative approach in the safe design for lateral capacity of piles is worth.

It is also suggested that the pile test shall be conducted adjacent to the boring area in order to study the test data in conjunction with boring data. The use of pile foundation for bridges with deep scours and without any possibility of socketing at the bottom needs serious examination and perhaps more research before adopting.

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Seismic Response for Cable-Stayed Bridge Pylon Foundation Considering Soil-Structure Interaction

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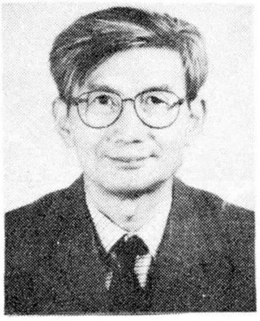

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SUMMARY

In this paper, soil-structure interaction was investigated in the simpler model called Single Pylon Model(SPM), and in order to attest the rationality of SPM, more elaborate model called Whole Bridge Model(WBM) was also used. The non-linearity of soil was dealt with equivalent linearity method in which the equivalent stiffness and equivalent damping ratio were calculated by special iterational program, the equivalent masses were calculated by the energy equivalent law. The foundation spring stiffness' of translation, rotation and coupling each other term are calculated by SPM. The pile foundations are substituted by SPM results in WBM. Using the seismic input motion of bed rock which have been proposed according to earthquake risk analysis, the acceleration history and the response spectrum at top of cap slab were evaluated by SPM which are used as the input spectra in WBM to evaluate the seismic response.

The natural periods and seismic responses of the bridge are calculated by the two proposed methods respectively. Those results by SPM are coincided favorably with those corresponding item by WBM. It is demonstrated the SPM method for seismic response analysis considering soil-pile-pylon interaction is worthy of continued study.

1 INTRODUCTION

The long-span (180m+312m+180m) cable-stayed bridge (Fig.1) is building at Wuhu City to cross Yangtze River. The pile foundation is adopted in this major bridge. The geological condition of the bridge is very complicated and every pylon's grounds are variant remarkably. The covering soil of the long pylon is 27m depth which is from -43m to -16m in altitude, while that of the short one is only 9.2m depth which is from -33.8m to -24m. It is obvious that the soil-pile-pylon interaction of each pylon is different. In order to calculate the seismic responses and dynamic behaviors of each pylon considering the interaction of soil-pile-pylon respectively, the simpler model called Single Pylon Model (SPM) is used. In order to attest the rationality of SPM, more elaborate model called Whole Bridge Model (WBM) is used also.

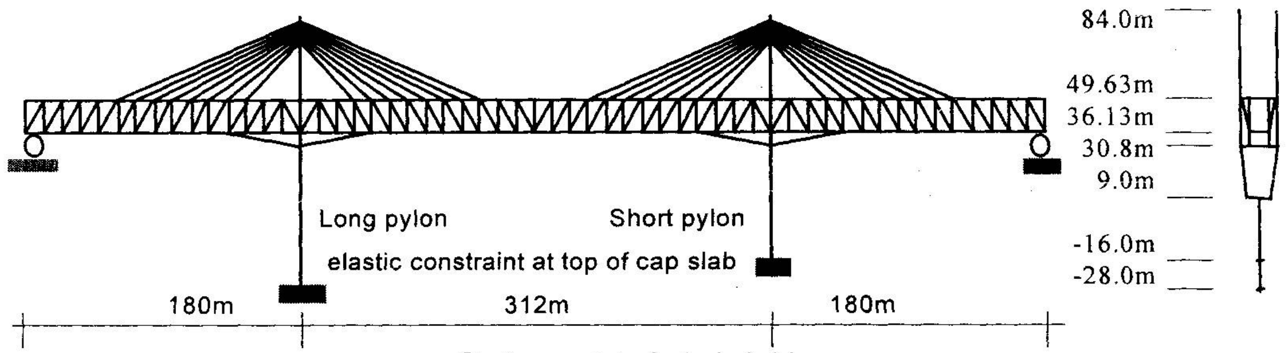


Fig.1 model of whole bridge

2 ANALYSIS METHOD

The analysis of soil-pile-pylon interaction in SPM mainly consists of two steps. The first is free field analysis of earthquake response and the second is the soil-pile-pylon interaction analysis in which the pile foundation elasticity-confined to the field is considered. The boundary conditions of the second are provided by the first.

2.1 Seismic response analysis of the free field

The assumptions about the soil are: The surface is horizontal; The soil in one layer is homogeneous; The soil is boundless. One-dimension soil column model is adapted to simulate the free field. The kinetics equation of the free field is:

$$\mathbf{M}^G\ddot{\mathbf{U}}^G + \mathbf{C}^G\dot{\mathbf{U}}^G + \mathbf{K}^G\mathbf{U}^G = -\mathbf{M}\mathbf{I}^G\ddot{u}_g$$

in which: $\mathbf{U}^G$ vector of seismic 3-dimensional response; $\ddot{u}_g$ acceleration of the base rock; $\mathbf{M}^G$ mass matrix with the diagonal elements $m_i^G = \frac{1}{2}(\rho_i h_i + \rho_{i+1} h_{i+1})$; $\mathbf{K}^G$ stiffness matrix with verticalital and horizontal elements respectively $K_{wi}^G = \frac{2G_i}{h_i}\left(\frac{1-\gamma_i}{1-2\gamma_i}\right)$ and $K_{ui}^G = \frac{G_i}{h_i}$; $\mathbf{C}^G$ Rayleigh damping matrix. In preceding formula : ρ_i, h_i, G_i, γ_i are mass per meter, height, shear modulus, Poisson ratio of the *i*th layer respectively.

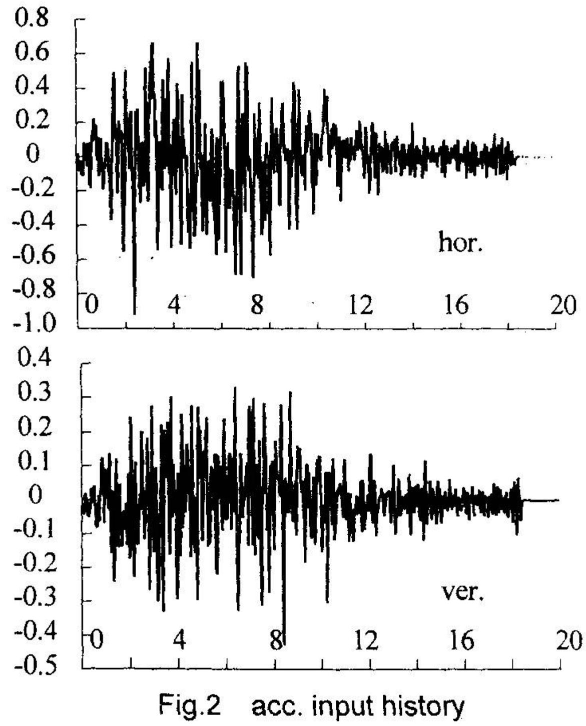


Fig.2 acc. input history

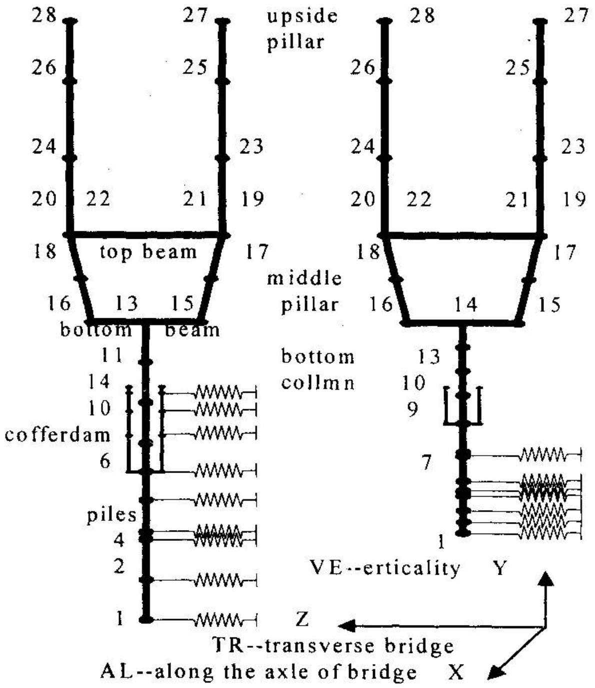
The non-linearity of soil is dealt with equivalent linearity method in which the equivalent stiffness and equivalent damping ratio were calculated by iterational. Special program is compiled for the purpose.

The seismic input motions of bed rock are provided by the Seismic Bureau of Anhui province, in which province the bridge is building. The probability of exceedence in 100 years is 10%. As the results of seismic risk analysis at bridge site, there are 12 inputs provided, which include 6 in horizontal and 6 in vertical respectively. One of six inputs in each direction is shown in Fig.2. Its maximum acceleration is 0.952m/s*s in horizontal and 0.436m/s*s in vertical respectively.

2.2 Analytical model

The group piles are simulated by a fictitious pile. The equivalent spring's stiffness at the bottom of cap slab caused by pile's support are computed according $K_{\varphi u} = \sum_{i=1}^{Q} x_i^2 \cdot k_{pi}$ in which: Q the total number of piles, k_{pi} the axial stiffness of the ith pile according to Sato assumption, x_i the coordinate of the ith pile. The SPM is shown in Fig.3. The characters of cross sections of the pylons are shown in table 1. The stiffness matrix is assembled by beam elements, and the mass matrix by lumped mass. In order to reduce the freedom for consider soil-pile-pylon interaction, the mass of main truss and other auxiliary is allocated reasonably with lumped mass in SPM. In order to ensure the comparability, the method of calculating the data in SPM is conformed with those in WBM.

Using those foundation spring stiffness proffered by SPM, the WBM is shown in Fig.1, in which the pylon is simulated by beam elements as well as main truss and auxiliary.



long pylon short pylon

Fig.3 Single pylon model

	section area	polar inertia moment	inertia moment		linear density
	m**2	m**4	m**4	m**4	t/m
single pile	7.069	7.952	3.976	3.976	18.732
long pylon fictitious pile	134.303	151.091	75.545	75.545	355.903
short pylon fictitious pile	120.166	135.183	67.593	67.593	318.440
cap slab	730.617	84957.0	42478.51	42478.5	1936.134
cofferdam	127.988	27158.2	13579.1	13579.1	339.169
long pylon	144.7	6060.9	7311.1	2889.4	383.46
bottom column	300.7	11652.0	11982.0	4756.7	796.96
short pylon	135.240	5517.0	6972.8	2562.0	358.545
bottom column	284.020	10003.0	11455.0	3945.0	752.653
bottom beam	68.340	550.360	738.824	205.02	181.101
middle pillar	34.700	212.610	92.510	445.370	91.955
top beam	16.900	78.740	67.826	48.348	44.785
	30.900	174.760	212.490	96.760	81.885
upside pillar	25.200	116.310	53.450	202.680	66.780

note : pylon elements E=0.35e11 pa, G=0.129e11 pa; pile elements E=0.31e11 pa ,G=0.122e11 pa

Table 1 The character of cross section in SPM

	long pylon			short pylon		
	AL	VE	TR	AL	VE	TR
joint of pylon and truss	1771	6833	3574	5597	7565	10179
top of pylon	3330.7	168.3	382.6	4916.0	168.33	548.57

Table 2 The allocated mass of main truss and other auxiliary in SPM (t)

2.3 Equation of Soil-Pile Interaction

The equations of motion considering the soil-pile-pylon interaction are:

$$\left(m_i^P + m_i^s\right)\cdot \ddot{u}_i + \sum_{j=1}^{n} c_{ij}^P \dot{u}_j + c_i^s \dot{u}_i + \sum_{j=1}^{n} k_{ij}^P u_j + k_i^s u_i = -m_i^P \ddot{u}_g + m_i^s \ddot{u}_i^G + c_i^s \dot{u}_i^G + k_i^s u_i^G$$

in which: the relative displacements of the pile and pylon $\{u\} = \{u_i\}$; the relative displacement of the soil $\{u^G\} = \{u_j^G\}; i = 1,\cdots,n$. Other parameters are shown in reference [1].

2.4 Equivalent Parameter of Soil-Pile Interaction

The equivalent horizontal stiffness between soil and the fictitious pile are calculated by Mindlin formula and Elasto Winkler assumption[1]. The equivalent vertical stiffness between soil and the fictitious pile are calculated according Sato assumption. The equivalent masses of the soil-pile interaction were calculated by the energy equivalent theory. At last the stiffness matrix and mass matrix of soil's equivalent effect are assembled according degree of freedom. Certainly the data of equivalent effect relevant to the pylon element is zero.

2.5 Seismic response spectra

The response spectra analysis method is used to calculate the seismic response in WBM. Using those 12 seismic input motions of bed rock, 18 acceleration histories--six respective in each of AL,VE,TR directions--at top of cap slab are analyzed by SPM to get the response spectra at the same location. Six response spectra are obtained in each direction by Duhamel integral. The envelope curve of the six response spectra is used as the input after studying the conformity of the six in each direction.

3 NATURAL PERIOD

The dynamic behaviors of the bridge are calculated by the two proposed models respectively. The natural periods are shown in the table 3. The SPM results are agreement with those of WBM.

	WBM	character of WBM	long pylon	short pylon	character of SPM
1	2.6843	truss,TR,symmetrical bending			
2	2.4420	two pylon, AL, floating	2.4369	2.55240	two pylon, AL
3	2.2460	truss and pylon,VE,symmetrical bending			
4	1.7088	long pylon and relevant beam,TR	1.73718		Long pylon TR
5	1.5718	short pylon and relevant beam,TR		1.68529	Short pylon TR
6	1.3606	short pylon and relevant beam,TR,torsion			
7	1.3446	long pylon and relevant beam,TR,torsion			
8	1.2982	short pylon ,TR		1.32631	Short pylon's limb TR
9	1.1666	long pylon ,TR	1.23953		Long pylon's limb TR
10	1.1624	truss and pylon,VE,anti-symmetrical bending			
11	1.1484	truss torsion			

Table 3 Period of bridge in WBM and SPM (s)

4 ANALYSIS RESULT

4.1 Stiffness Coefficient of Cap Slab of Pile Foundation

The foundation spring stiffness' of translation, torsion and coupling each other term that are calculated by SPM are shown in table 4.

These springs' coefficients are the constraint conditions at the cap slab to alternate the pile foundations in the WBM.

		long pylon	short pylon
length of piles	m	30	20
number of piles		19	17
diameter of piles	m	3.0	3.0
VE translation k_y	kN/m	1.683E8	4.085E8
TR translation k_z	kN/m	0.116E9	0.439E9
TR coupling k_{z,φ_x}	kN/rad	-1.730E9	-7.503E9
TR rotation k_{φ_x}	kN*m/rad	34.72E9	151.47E9
AL translation k_x	kN/m	0.115E9	0.439E9
AL coupling k_{x,φ_z}	kN/rad	1.718E9	7.503E9
AL rotation k_{φ_z}	kN*m/rad	34.86E9	151.47E9

table 4 The constrain coefficients at cap slabs

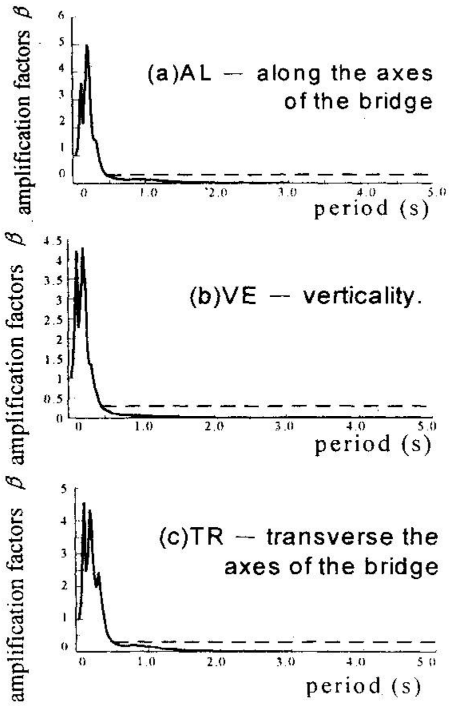


Fig.4 Envelope curves (response spectra at the top of cap slab in long pylon)

4.2 Envelop Curve of the Response Spectra

The envelope curves of the response spectrum at the top of cap slabs which are shown in Fig.4 and table.5 are calculated by SPM.

Those spectra are the input spectrum in the WBM.

		displacement	a_{max}	β_{max}	$\beta_{max} \times a_{max}$	a_{max} at period	β_{min} start period	$\beta_{min} \times a_{max}$
		mm	m/(s*s)		m/(s*s)	s	s	m/(s*s)
long pylon	AL	3.717	1.4476	5.01996	7.26689	0.20	0.46	0.4343
	VE	2.785	1.0735	4.29193	4.60738	0.20	0.45	0.3221
	TR	3.240	1.2041	4.55865	5.4888	0.12	0.52	0.3612
short pylon	AL	1.073	0.4034	4.22372	1.7038	0.28	0.56	0.1210
	VE	0.389	0.2784	4.02578	1.1208	0.08	0.44	0.0835
	TR	1.180	0.4103	5.02317	2.0610	0.28	0.60	0.1231

Note : 1. The probability of exceedence in 100 years is 10%, damp ratio is 0.05; 2. Refer to 《Highway Engineering Aseismic Design Code》 β_{min}=0.30, that is when the natural period is greater than the data in the last row of the table, amplification factor β is 0.30 .

Table 5 envelope curves of response spectrum at the top of cap slabs

4.3 Seismic responses

The section forces and displacement of the bridge are calculated by the two proposed models respectively. The results were shown in the table 6. Clearly the SPM results are agreement with those of WBM

		SPM 4 modes	WBM 10 modes		WBM 30 modes	SPM 8 modes	WBM 54 modes		WBM 70 modes
the displacements of middle joint of the main truss and top joint of each pylon									
main truss TR	cm		9.7957		9.8016		9.8022		9.8022
main truss AL	cm		7.8944		7.8946		7.9027		7.9033
main truss VE	cm		3.1576		3.2597		3.2632		3.2711
long pylon TR	cm	5.6203	4.1317		4.1438	5.67	4.1599		4.16
long pylon AL	cm	9.4638	7.9649		7.9677	9.46	7.9743		7.9742
short pylon TR	cm	5.6542	3.7510		3.7656	5.71	3.7729		3.773
short pylon AL	cm	9.8567	8.3616		8.3639	9.82	8.3655		8.4059
the section forces of bottom joint of each pylon: M is moment, Q is shear force, N is axial force									
long pylon TR M	t-m	43428	44864	3.31%	46346	50838	47827	5.92%	47826
long pylon TR Q	t	550.05	617.42	12.24%	721.64	1957.2	1776.9	9.25%	1777.6
long pylon AL M	t-m	81747	78402	4.27%	79376	84981	79692	6.22%	79685
long pylon AL Q	t	898.73	989.53	10.1%	1085.6	1208.5	1268.0	4.73%	1263.1
long pylon N	t	.06513	114.55		282.14	650.89	371.75		621.06
short pylon TR M	t-m	43140	40500	6.12%	43087	50150	45722	8.83%	45723
short pylon TR Q	t	620.80	618.52	0.037%	774.44	915.49	880.95	3.77%	880.95
short pylon AL M	t-m	88091	91411	3.63%	91495	90578	93852	3.48%	103510
short pylon AL Q	t	1101.7	1303.5	15.49%	1387.2	1279.5	1504.8	14.97%	2851.1
short pylon N	t	.06691	121.73		272.24	481.12	334.02		471.00

Table. 6 section forces and displacements of seismic response in WBM and SPM

5 CONCLUSIONS

Considering this study, the following conclusion can be made:

1. Because the bridge is so major and the geological conditions of each pylon are so complex, the effects of dynamic soil-pile-pylon interaction should be considered. It is possible and expedient to make these considerations into reality in SPM. The elastic constraints at the top of cap slab are calculated by SPM for replacing pile foundation in WBM.

2. It is convenient to calculate the response spectra at the top of cap slab by SPM. The envelope curves of those spectra are used as the input spectra in WBM to evaluate the seismic responses.

3. The section forces and displacements of seismic response are evaluated by SPM and WBM respectively. Those results by SPM are coincided favorably with those identical items by WBM. It is demonstrated the SPM method for seismic response analysis considering soil-pile-pylon interaction is worthy of continued study.

Note: In the whole paper direction symbol (which has been shown in Fig.3) : AL — along the axes of the bridge; TR — transverse the axes of the bridge; VE — verticality.

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SAFE LOAD FROM DEFICIENT PILE LOAD TEST DATA

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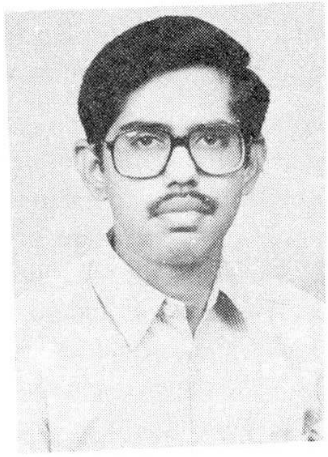

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SUMMARY

Initial pile load tests are conducted in major piling projects. The Indian Standard IS:2911 (Part 4)-1985, specifies two criteria for the determination of safe load of a pile. The criteria are in terms of the loads corresponding to total settlements of 12 mm and 10% of the pile diameter. Therefore, to apply both the criteria the settlement of the pile should not be not less than 10% of the pile diameter. When the load-settlement data is deficient, it is desirable to make a reasonably good estimate of the loads corresponding to each criterion by a proper interpretation of the available data. The paper explains the hyperbolic method and the modified hyperbolic method that can be used for this purpose. The paper presents the results of the analyses of data from twnty four pile load tests by using these methods. The results show that for piles transmitting more than 65% of their load through skin friction, the loads corresponding to each criterion of the Indian Standard and the load-settlement curve can be predicted reasonably well by using the hyperbolic method and the modified hyperbolic method, if the load-settlement data is available up to a minimum settlement of 5% of the pile diameter.

1 INTRODUCTION

The Indian Standard Code of Practice IS:2911 (Part 4)-1985 {1} specifies the criteria for the safe load of piles tested in compressive, tensile, and lateral loading. According to the Indian Standard, the safe load of a pile in compression is the smaller of the following two values:

Criterion 1: Two-thirds of the final load at which the total displacement attains a value of 12 mm unless otherwise required in a given case on the basis of the nature and the type of the structure in which case, the safe load should be corresponding to the stated total displacement permissible.

Criterion 2: 50% of the final load at which the total displacement is equal to 10% of the pile diameter in case of uniform diameter piles and 7.5% of bulb diameter in case of under-reamed piles.

For several reasons, often even in the initial load tests, the pile is not subjected to loads that would cause a settlement of 10% of the pile diameter. If the settlement of the pile is more than 12 mm, then the safe load can be estimated by using only the first criterion. When the load-settlement data is deficient, it is desirable to make a reasonably good estimate of the loads corresponding to each criterion by a proper interpretation of the available data. The paper explains analytical procedures which can be used for this purpose. The Australian Standard AS 2159-1995 {2} specifies the acceptance criteria in terms of gross settlement and net settlement. The analytical procedures were used for the Australian Standard criteria also. The paper explains the salient details of the study with respect to the Indian Standard only. For more information, reference can be made to another paper of the author cited at the end {3}.

2 ANALYSIS OF PILE LOAD TEST DATA

To develop analytical procedures, the load-settlement data and other details of pile load tests, in which the settlement exceeded 10% of the pile diameter, were collected from twnty four pile load tests on straight shaft piles. In one test, the pile diameter was 1070 mm, and in other tests it varied from 400 to 800 mm. The pile length varied from 13 to 27 m. Seven of the pile load tests were on driven *cast-in-situ* piles in cohesive soils, nine were on driven *cast-in-situ* piles in noncohesive soils, and six were on bored *cast-in-situ* piles in noncohesive soils. All tests were slow maintained load tests and eight of them were cyclic tests.

Each pile load test data was first scrutinized for consistency and errors. The load-settlement curve is generally a nonlinear curve with the slope - the ratio of incremental settlement to incremental load - increasing continuously with load. A seating error is indicated if at the beginning of the curve there is a decrease in the slope. The initial range of the data was examined. If there was a seating error, the correction in settlement required to make the slope more or less linear in this range was identified. The correction was applied to the raw data. The load-settlement curve was drawn using the corrected data. From this, the loads corresponding to the two criteria of the Indian Standard Code were determined. The loads corresponding to the first and second criteria were designated as $Q_{1\text{-}IS}$ and $Q_{2\text{-}IS}$, respectively. In nineteen pile load tests, including the 1070 mm diameter pile, the safe load was governed by the second criterion, that is $Q_{1\text{-}IS} > Q_{2\text{-}IS}$. This indicates that the safe load estimated from the first criterion alone can be unconservative. The value of the ratio $Q_{1\text{-}IS}/Q_{2\text{-}IS}$ varied from 0.823 to 1.243 with an average of 1.093. The standard deviation and 95% confidence interval of the ratio were 0.111 and 0.044, respectively.

Different relationships between load and settlement were considered. For each relationship, the expressions for the load corresponding to each criterion were obtained. The data were artificially made deficient by choosing the data only up to a particular value of settlement. From a regression analysis of this data, the parameters of the expressions were obtained. The rigor of the regression analysis was tested from the correlation coefficient, the F-test, and the t-test. The loads corresponding to the two criteria of the Indian Standard were estimated using the corresponding expressions. The load-settlement curve was also predicted. The analysis was repeated for different ranges of the load-settlement data.

An assumed relationship was considered to be satisfactory if the following conditions were satisfied:

a) the load estimated for each criterion compared well with the respective actual load,

b) the estimated values of the safe loads compared well with the actual values,

c) the criteria for the estimated safe loads matched with the criteria that gave the actual safe loads,

d) the predicted and actual load-settlement curves matched well, and

e) the relevant statistical tests were satisfied.

Only the hyperbolic method and the modified hyperbolic method satisfied all these conditions. Therefore, the results of only these two methods are presented in the paper.

3 HYPERBOLIC METHOD

The expressions for the loads corresponding to the Indian Standard criteria are

$$Q_{h1-IS} = \frac{8}{b + 12m} \quad \text{kN} \tag{1}$$

$$Q_{h2-IS} = \frac{0.05D}{b + 0.1mD} \quad \text{kN} \tag{2}$$

$Q_{h1\text{-}IS}$ and $Q_{h2\text{-}IS}$ are the loads given by the hyperbolic method corresponding to the first and second criteria of the Indian Standard, respectively. b and m are constants obtained from linear regression analysis of the data in the (s, s/Q) coordinates system. s is the settlement of pile at pile load Q. The units of s and Q are mm and kN, respectively. D is the pile diameter in mm.

4 MODIFIED HYPERBOLIC METHOD

Rollberg {4} suggested that the load-settlement curve is a hyperbola up to a settlement s_o, beyond which it is a straight line as shown in Fig. 1. s_o is given by an empirical relationship and constants that are dependent on the type of soil and the method of pile installation.

The expressions for the loads corresponding to the Indian Standard criteria are

$$Q_{m1-IS} = \frac{8}{b + 12m} \quad \text{kN} \qquad s_o \geq 12 \text{ mm} \tag{3a}$$

$$Q_{m1-IS} = \frac{2}{3}(c + 12i) \quad \text{kN} \qquad s_o \leq 12 \text{ mm} \tag{3b}$$

$$Q_{m2-IS} = \frac{0.05D}{b + 0.1mD} \quad \text{kN} \qquad s_o \geq 0.1D \text{ mm} \tag{4a}$$

$$Q_{m2-IS} = \frac{1}{2}(c + 0.1iD) \quad \text{kN} \qquad s_o \leq 0.1D \text{ mm} \tag{4b}$$

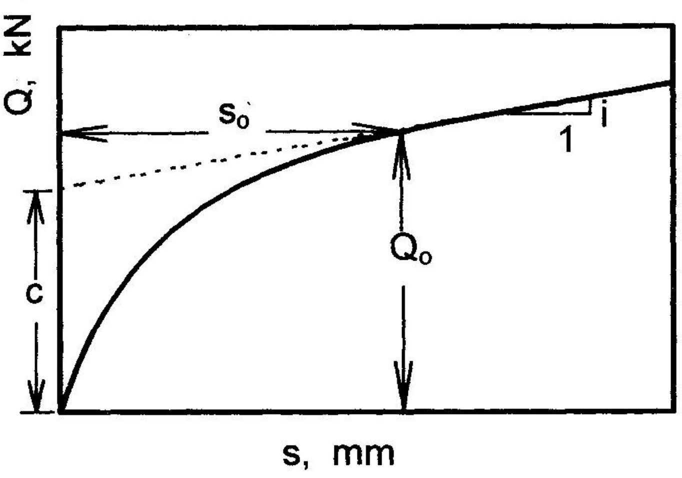

Fig. 1 Modified Hyperbola

$Q_{m1\text{-}IS}$ and $Q_{m2\text{-}IS}$ are the loads given by the modified hyperbolic method corresponding to the first and second criteria of the Indian Standard, respectively. i is the slope of the straight line portion of the load-settlement curve beyond s_o and c is its intercept with the load axis.

5 RESULTS

5.1 Comparison of the Estimated and Actual Loads

The loads estimated by using the hyperbolic method and the modified hyperbolic method were compared with their respective actual loads by computing the ratio of the estimated load to the actual load. The closer the value of this ratio is to unity, better will be the theoretical estimate of the load. The trend of variation of the different ratios with s/D was studied and the upper and lower limits of the variation were determined. Table 1 gives the expressions for the upper and lower limits for the different ratios. The respective ranges of s/D in which the error in the different estimated loads is less than 10% and 5%, and the mean, standard deviation, and 95% confidence interval of the ratios over these ranges of s/D were determined. These results are presented in Table 2. For the same data, the results from the modified hyperbolic method were slightly better than those from the hyperbolic method.

5.2 Load-Settlement Curves

The load-settlement curves predicted from different ranges of deficient data were compared with the actual load-settlement curves. The minimum value of s/D required to predict the load-settlement curve reasonably well ranged from as low as 0.82% to as high as 10%. Still, in majority of piles the required s/D value was very low. In fourteen piles it ranged between 0.82% and 3%, in five piles between 3% and 5%, and in four piles above 5%. The average, standard deviation, and 95% confidence interval of the minimum required s/D value were 3.43%, 2.43%, and 0.97%, respectively. In modified hyperbolic method, it was observed that for bored piles the constants recommended by Rollberg for driven piles instead of bored piles should be used in the analysis.

Table 1: Lower and upper limits for load ratios from the hyperbolic and modified hyperbolic methods

Ratio, y	Limit	Expression
$Q_{h1\text{-}IS}/Q_{1\text{-}IS}$	Lower	$y = -64.4x^2 + 9.1086x + 0.7204$
	Upper	$y = -485494x^5 + 181705x^4 - 26212x^3 + 1808x^2 - 58.472x + 1.7566$
$Q_{h2\text{-}IS}/Q_{2\text{-}IS}$	Lower	$y = -307.05x^4 + 468.83x^3 - 109.9x^2 + 9.9851x + 0.64$
	Upper	$y = 21281x^4 - 7205.8x^3 + 889.62x^2 - 47.092x + 1.8906$
$Q_{m1\text{-}IS}/Q_{1\text{-}IS}$	Lower	$y = -9384.7x^4 + 3115.8x^3 - 367.33x^2 + 17.555x + 0.7062$
	Upper	$y = -688985x^5 + 245552x^4 - 33567x^3 + 2184.9x^2 - 66.395x + 1.7867$
$Q_{m2\text{-}IS}/Q_{2\text{-}IS}$	Lower	$y = 1.2687x^{0.1081}$
	Upper	$y = 154336x^5 - 18178x^4 - 4001.3x^3 + 827.58x^2 - 49.905x + 1.9933$
y = ratio of the estimated load to the actual load; $x = s/D$ (*not* in per cent)		

Table 2: Statistical values for the load ratios from the hyperbolic and modified methods

Quantity	$Q_{h1\text{-}IS}/Q_{1\text{-}IS}$	$Q_{h2\text{-}IS}/Q_{2\text{-}IS}$	$Q_{m1\text{-}IS}/Q_{1\text{-}IS}$	$Q_{m2\text{-}IS}/Q_{2\text{-}IS}$
Over the range of *s/D* in which the error in the estimated load is within 10%				
Range of *s/D*%	≥ 2.5	≥ 3.5	≥ 1.7	≥ 2.6
Average	1.002	0.969	1.017	0.980
Standard Deviation	0.0386	0.0511	0.0482	0.0887
95% Conf. Interval	0.0095	0.014	0.012	0.0262
Over the range of *s/D* in which the error in the estimated load is within 5%				
Range of *s/D*%	2.5 - 5.5	≈ 10	2.2 - 6	≈ 10
Average	1.013	0.992	1.011	1.006
Standard Deviation	0.0254	0.0106	0.020	0.0156
95% Conf. Interval	0.0085	0.0041	0.0076	0.0074

5.3 Effect of Irregularities and Seating Error in the Actual Load-Settlement Curve

The results obtained from the hyperbolic and modified hyperbolic methods were not sensitive to the irregularities in the actual load-settlement curve. However, the effect of the correction for seating error on the results was significant. Even a small correction, not only improved the predictions significantly but also reduced the amount of data required to do that. The initial range of the observed load-settlement data should therefore be scrutinized carefully to determine the need for any correction to be done in the data.

5.4 Components of Load Transfer

The analysis for the nature of load transfer showed that except for one pile in all the other piles the skin friction was the predominant component. In many piles the skin friction was more than two-thirds of the load acting on the pile.

5.5 Illustrative Example

At Numaligargh, Assam, India, a 500 mm diameter bored cast-in-situ pile of length 19.175 m was subjected to a maximum load of 1207 kN at which the pile settlement was 21.61 mm ($s/D = 4.32\%$). Figure 2 shows the actual load-settlement curve and that predicted by the modified hyperbolic method. The best estimate for the safe load of the pile is obtained by the modified hyperbolic method. According to this method, the safe load is governed by the second criterion and is in the range of 639-665 kN.

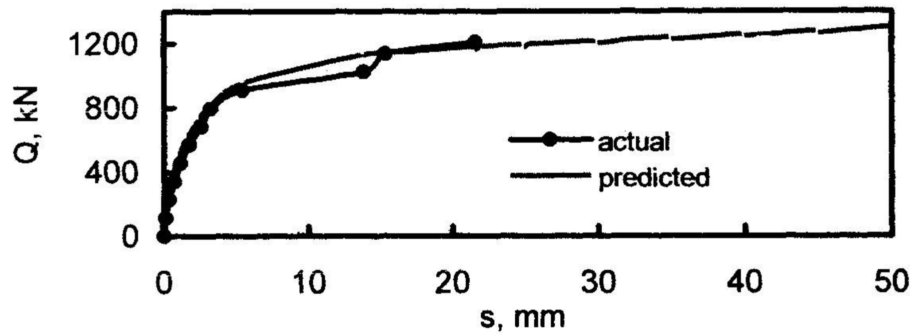

Fig. 2 Actual and predicted load-settlement curves for Numaligargh pile

6 CONCLUSIONS

The loads corresponding to the two criteria of the Indian Standard and the load-settlement curve of piles that transmit more than 65% of their load through skin friction could be predicted reasonably well by the hyperbolic and the modified hyperbolic methods, if the load-settlement data is available up to a minimum settlement of 5% of the pile diameter. For the same data, the modified hyperbolic method gives slightly better results than the hyperbolic method. The results are not sensitive to the irregularities in the actual load-settlement curve but to the correction made for seating error.

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EFFECT OF PILE CAP FLEXURAL RIGIDITY ON THE BEHAVIOR OF BRIDGE FOUNDATIONS

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SUMMARY

In the design of reinforced concrete bridge foundations, most of the pile caps are assumed to be rigid; neglecting the effect of the pile caps' rigidities. Also, the piles are considered to be acting as rigid supports for the pile caps; without taking into account their axial stiffness.

The main objective of the present research work is to emphasize the effects of the pile cap flexural rigidity and the piles' axial stiffness on distributing the column loads on the piles; and consequently the global behavior of the bridge foundation.

A reinforced concrete pile cap designed as a typical pier foundation for a multi-span highway bridge is analysed by the finite element method considering the cracked modulus of elasticity of concrete. In this study, the thickness of the pile cap is changed in order to study the effect of its flexural stiffness on the behavior of the foundation. The pile cap is subjected to the different load cases specified in the AASHTO code {1}; including the seismic loads obtained from the dynamic analysis for seismic performance category "B". Also, the piles are modeled as elastic springs taking their axial stiffness into account.

The results of the present research work have been used to formulate some recommendations concerning the design of the pile caps and the distribution of the pile loads.

1 INTRODUCTION

In the common design of reinforced concrete bridge foundations, the pile caps are assumed to be rigid in the determination of the pile loads. Also, the piles are considered to act as rigid supports. This research work studies the effects of the pile cap rigidity and the axial stiffness of the piles on distributing the column loads on the piles; and consequently the design of the pile cap itself. These effects are more significant in the cases of pile caps which are subjected to high loads and moments, such as those of multi-span bridge foundations, especially in the seismic load cases.

A typical pier foundation for a multi-span highway bridge, about 400m long, has been chosen to study the effect of the pile cap flexural rigidity, and the axial stiffness of the piles on the overall behavior of the bridge foundation. The intermediate spans of the bridge are 35m, while the end spans are 25m. The width of the bridge is 17.0m, including four highway lanes.

The typical cross section of the bridge consists of a double vent prestressesd box girder, (Fig. 1). The bridge deck is supported on central piers. In the longitudinal direction of the bridge, the three intermediate piers are monolithic with the deck; while the rest of the piers have guided bearings to allow for the longitudinal deck movement due to temperature variation, creep and shrinkage. The piers have different heights varying between 5m and 12m.

A detailed finite element model was prepared for the bridge considering the highway moving loads according to the AASHTO specifications {1}, in addition to the prestressing loads, the temperature effects, and the seismic loads obtained from the dynamic analysis. Another finite element model was prepared for the typical pile cap of the monolithic piers; which resists the most critical loads from the temperature movements and the seismic forces, (Fig. 2).

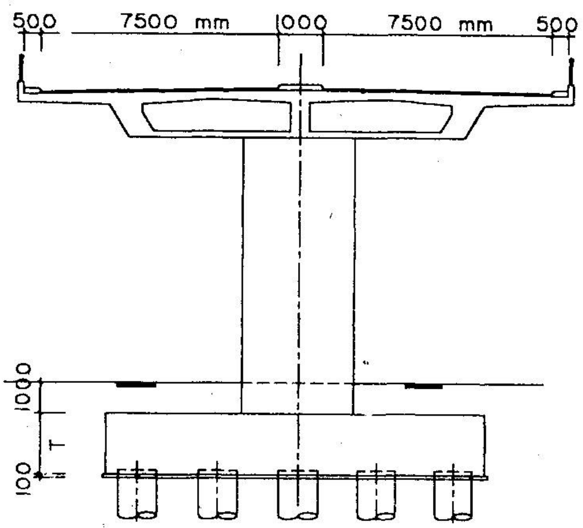

Fig. 1 Typical bridge cross section

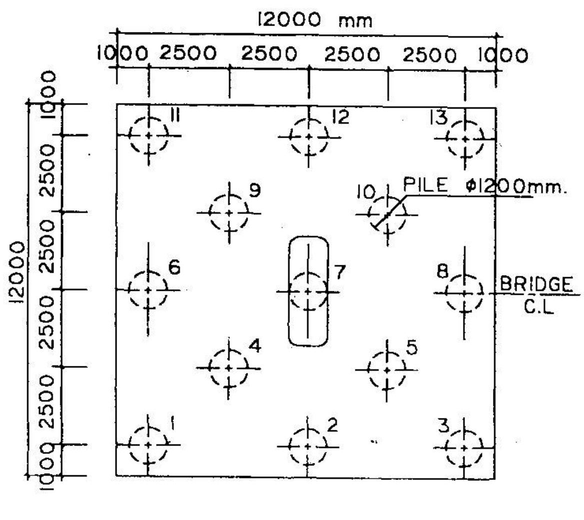

Fig. 2 Typical pile cap plan

2 DESCRIPTION OF THE PILE CAP AND THE ACTING LOADS

The studied pile cap size is 12 x 12m and is supported on thirteen piles. The length of each pile is 24m and its diameter is 1.2m with a vertical capacity of 3000 KN. This capacity can be increased by 33% in the seismic load cases. The box girder bridge is loaded as per AASHTO code {1}. The dead load carried by the typical pier is 12750 KN. The maximum reaction on the pier due to the live loads (4 lanes) is about 4000 KN; while that due to the eccentric live loads is about 2600 KN, in addition to a transverse moment of about 12000 KN.m.

The bridge was designed according to the AASHTO specifications for seismic design {2}, and was classified as seismic performance category "B". Thus, the acceleration coefficient was considered as 0.19g and the elastic moments due to the earthquake forces were determined independently along both the longitudinal and transverse directions of the bridge using the SAP2000 program {3}. The seismic moments for the critical pier were found to be 50000 KN.m and 60000 KN.m in the longitudinal and transverse directions, respectively. However, it was found that the moment obtained in the longitudinal direction exceeded the elastic moment capacity of the pier. Accordingly, the elastic moment in this direction was divided by the appropriate response modification factor for the footings; and a plastic hinge design was considered.

The critical case of loading for the studied pile cap was found to be the case of earthquake in the transverse direction. Thus, 100% of the obtained transverse moment was considered, in addition to 30% of the obtained modified moment in the longitudinal direction. This load combination was taken according to AASHTO load combinations to account for the directional uncertainty of earthquake motions and the simultaneous occurrences of earthquake in the two principal directions

3 FINITE ELEMENT MODELING AND PARAMETRIC STUDY

The pile cap was analysed using a three dimensional finite element model by the SAP90 program {4}. The model consisted of 625 nodes and 576 elements. In this analysis, the reinforced concrete pile cap was assumed to be a linearly-elastic material with a modulus of elasticity of 2.1E7 KN/m^2 and a Poisson's ratio of 0.2. To consider the effects of concrete cracks on the pile cap stiffness, the modulus of elasticity was reduced by 25%. Since the flexural stiffness of a pile cap is a function of its dimensions (depth to width ratio); the pile cap was analyzed using different depths (1.0, 1.5, 2.0, 2.5, 3.0 and 5.0 m). All of the thirteen piles were modeled as elastic spring supports considering their axial stiffness. The elastic spring constants were taken as 742200 KN/m for the compression piles, and 371000 KN/m for the tension piles. This reduced value for the tension piles was due to the fact that the pile displacement associated with tension was larger than that in compression under the same amount of axial load.

4 RESULTS OF THE PARAMETRIC STUDY

4.1 Pile Cap Deflections

A comparison is made between the deformed shapes of the studied pile cap under the applied loads for 1m and 3m thickness in (Fig. 3 and 4), respectively. It is clear that for the pile cap with 1m thickness, a flexible behavior is obtained with a maximum deflection value of about 1.0 cm. Thus, the assumption of a rigid footing becomes erratic for this thickness. On the other hand, the model with 3m pile cap thickness shows a rigid behavior; with a maximum deflection of about 0.43 cm. The same deflection value was obtained for the 5m thickness model. This considerable difference in the deformed modes and the obtained deflection values strongly affect the distribution of the applied loads on the supporting piles.

4.2 Moments Acting on Pile Caps

The values and patterns of the bending moments M_{11} and M_{22} of the studied pile cap in the two principal directions for 1m and 3m thickness are illustrated in (Fig. 5 to 8), respectively. From these figures, it can be concluded that:

- The negative moment variations between the 1m and 3m thickness pile caps can attain 28%; while positive moment variations can attain 19%.

- The maximum moments obtained from the model with 1m thickness are less than those obtained from the 3m thickness model. This proves that, the flexible pile cap reduces the loads on the edge and corner piles and consequently increases the loads on the interior ones. This may lead to critical cases for the loading capacities of the interior piles.
- The values of moments obtained from the computer models are much higher than those calculated by the rigid analysis. Also, in the computer models, the moments are concentrated near the column; while in the rigid analysis the moments are distributed along the width of the pile cap. This may cause a critical situation for the safety of the pile cap design, as obtained from models having small thickness between 1m and 2m.

4.3 Pile Reactions

The ratio of the pile loads obtained from the finite element analysis to those obtained from the analysis of the pile cap as a rigid footing ($P_{F.E}/P_{Rigid}$) versus the pile cap thickness; are shown in (Fig. 9 to 11) for the different locations of the piles: corner, edge, and interior piles, respectively. In these curves, the pile cap own weight and weight of fill have been excluded in order to emphasize the effect of the studied parameters on distributing the pier loads on the supporting piles. Table (1) shows the values of the pile loads both for a rigid footing analysis as well as for the results of the present finite element analysis. The negative load values indicate tension forces on the piles. From these figures and table, the following can be summarized:

- For corner piles, the ratio between the pile reaction obtained from the finite element analysis and that obtained from the rigid analysis can reach 9%, 72%, 90% and 98% for the models with 1m, 2m, 3m and 5m thickness, respectively. This large variation in the obtained reactions emphasizes the effect of the pile cap thickness and rigidity on distributing the applied loads.
- For edge piles, the ratio between the pile reaction obtained from the finite element analysis and that obtained from the rigid analysis can reach 1%, 85%, 95% and 99% for the models with 1m, 2m, 3m and 5m, respectively. For pile No. "2", the pile reaction obtained from the rigid analysis is (-734 KN), while that obtained from the model with 1m thickness is (-1730 KN). Upon the addition of the pile cap own weight and weight of fill, the aforementioned values reach (-257 KN) and (-980 KN), respectively. This tensile force (-980 KN) exceeds the own weight of the pile (680 KN) and represents a critical case for the pile.
- For interior piles, the ratio between the pile reaction obtained from the rigid analysis and that obtained from the finite element analysis can reach 11%, 58%, 90% and 99% for the models with 1m, 2m, 3m and 5m thickness, respectively. For pile No."10", the pile reaction obtained from the rigid analysis is (2052 KN); while that obtained from the model with 1m thickness is (4634 KN) and reaches (5158 KN) when taking into account the pile cap own weight and weight of fill. This value exceeds the pile capacity in seismic cases (4000 KN).

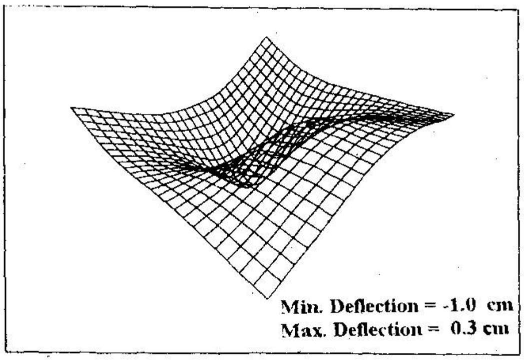

Fig. 3 Deformed shape of 1m thick. model

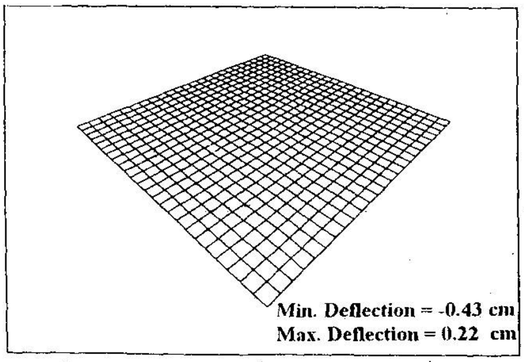

Fig. 4 Deformed shape of 3m thick. model

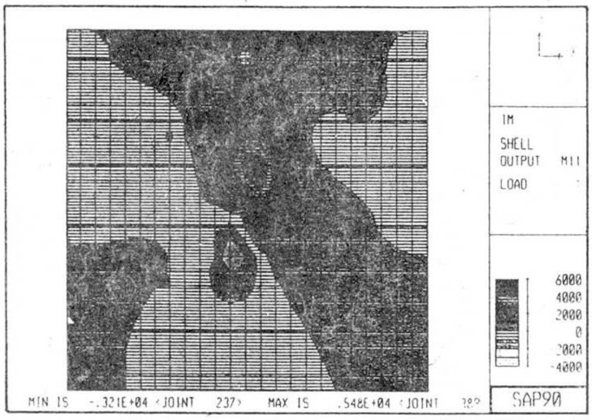

Fig. 5 M_{11} of 1m thickness model

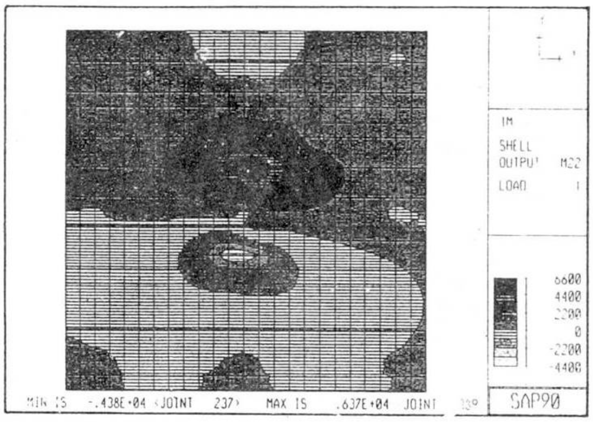

Fig. 6 M_{22} of 1m thickness model

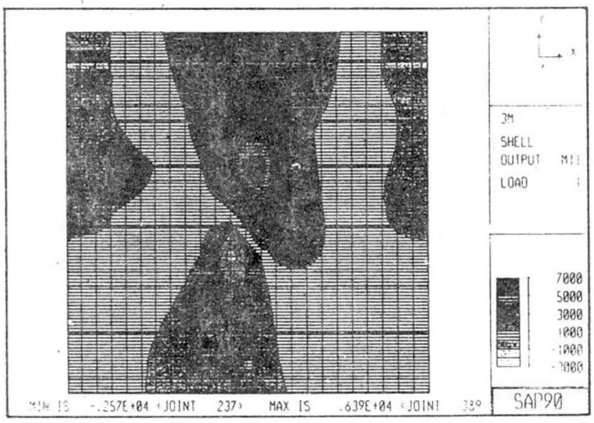

Fig. 7 M_{11} of 3m thickness model

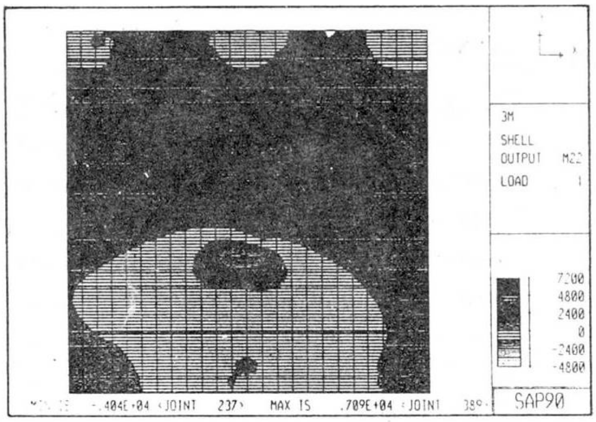

Fig. 8 M_{22} of 3m thickness model

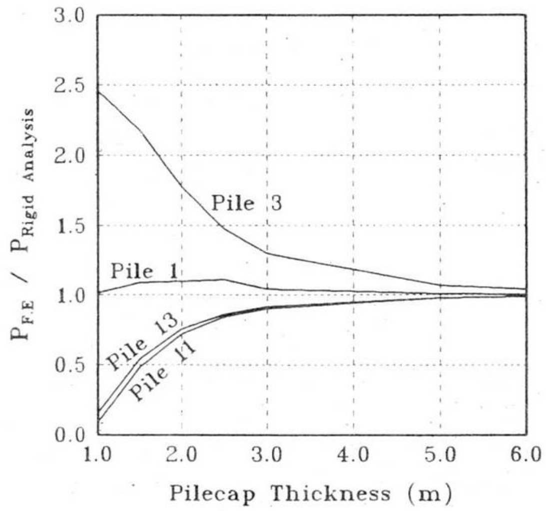

Fig. 9 ($P_{F.E}/P_{Rigid}$) ratio for corner piles

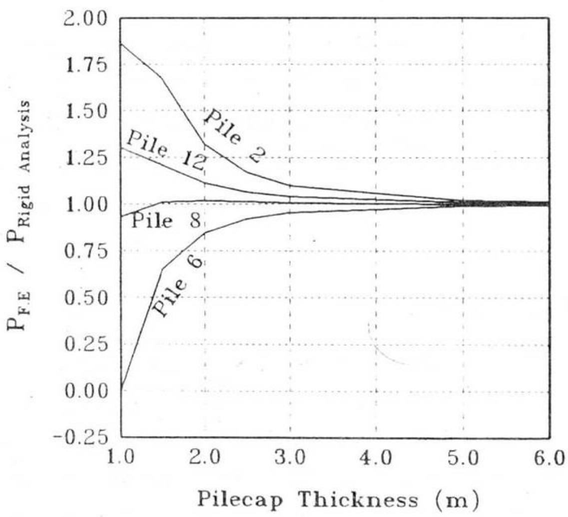

Fig. 10 ($P_{F.E}/P_{Rigid}$) ratio for edge piles

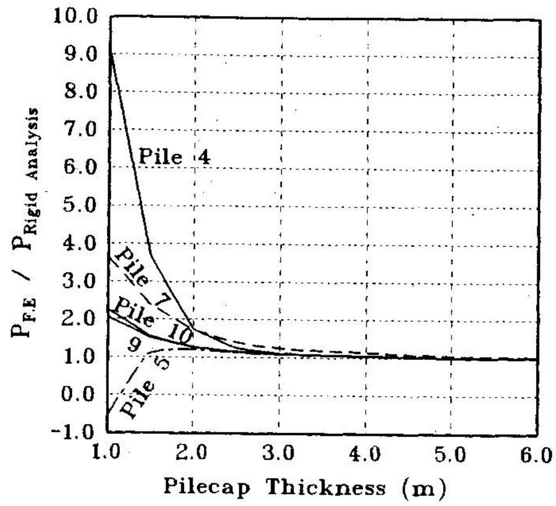

Fig. 11 ($P_{F.E}/P_{Rigid}$) ratio for interior piles

Pile No.	1m		2m		3m		5m		Rigid Analysis
	$P_{F.E}$	$P_{F.E^*}$	$P_{F.E}$	$P_{F.E^*}$	$P_{F.E}$	$P_{F.E^*}$	$P_{F.E}$	$P_{F.E^*}$	
1	-1180	-692	-1273	-562	-1213	-204	-1175	402	-1162
2	-1370	-982	-970	-208	-807	227	-750	837	-734
3	-750	-323	-540	170	-396	613	-327	1250	-305
4	-835	-482	-158	618	-101	940	-92	1496	-91
5	-186	418	410	1187	370	1412	346	1934	338
6	-4	523	468	1230	527	1561	547	2132	552
7	3579	4199	1690	2487	1223	2274	1035	2627	981
8	1315	1786	1435	2179	1420	2456	1412	2998	1410
9	3348	3890	2058	2834	1770	2811	1657	3245	1624
10	4634	5158	2626	3402	2240	3283	2094	3683	2052
11	200	579	1634	2345	2047	3057	2216	3793	2266
12	3514	4004	3002	3763	2800	3835	2719	4305	2695
13	483	862	2366	3077	2864	3874	3064	4641	3123

$P_{F.E^*}$ includes own weight of pile cap and weight of fill

Table 1 Pile Reactions for different pile cap thick.

5 CONCLUSIONS

- The design of the pile cap as a rigid footing is acceptable when the ratio (L/T) does not exceed (2.4); where (L) is the distance between the pier centerline and the center of the furthermost corner pile and (T) is the thickness of the pile cap.
- If the ratio (L/T) exceeds (2.4), a flexible behavior is expected and a detailed finite element analysis is required for the design of the pile cap.
- Flexible pile caps reduce the pile reactions on corner and edge piles and increase the loads on the interior ones. This may cause tensile forces on the outer piles; and may over-load the interior piles beyond their load carrying capacities.
- The flexural rigidity of the pile cap strongly affects the obtained deformed shapes of the footing and consequently affects the distribution of the pile loads.
- The rigidity of the pile cap considerably affects the obtained straining actions of the pile cap

6 NOTATIONS

L	:	The distance between the pier centerline and the center of the corner pile.
M_{11}, M_{22}	:	Pile cap moments in the longitudinal and transverse directions, respectively.
$P_{F.E}$	:	Pile load obtained from the finite element analysis.
P_{Rigid}	:	Pile load obtained from the analysis of the pile cap as a rigid foundation.
T	:	Pile cap thickness.

REFERENCES

1 AASHTO, Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials, Interim Specifications 1994.

2 AASHTO, Standard Specifications for Highway Bridges - Seismic Design, American Association of State Highway and Transportation Officials, Interim Specifications 1994.

3 SAP2000, Integrated Finite Element Analysis and Design of Structures, Computers and Structures Incorporation - Berkeley, California, USA.

4 SAP90, Computer Software for Structural and Earthquake Engineering, Computers and Structures Incorporation - Berkeley, California, USA.

Foundations of the Jamuna Bridge – Design and Construction

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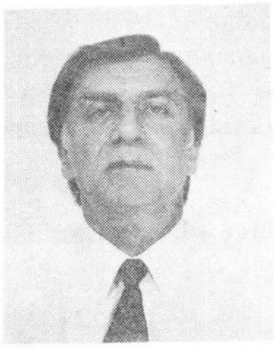

As Chief Bridge Engineer, Joe Barr has been closely involved with the Jamuna Bridge project from the early studies through construction. His areas of specialist expertise include segmental concrete bridges, caisson and piled foundations, seismic engineering, risk and reliability analysis, procurement of design-and-build bridge projects, and expert technical opinion on concrete structures. He is the author of various professional guidance publications, including bridge inspection, design for buildability, integral bridges and seismic engineering.

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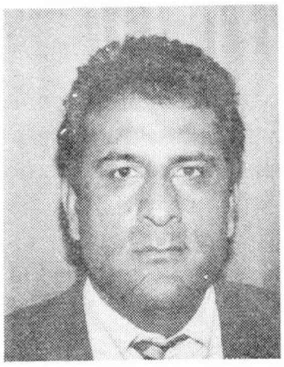

Abdul Farooq has been involved with the project for over 10 years, initially as a foundation design engineer in the engineering feasibility study of the project. Subsequently he was involved with the preparation of tender documents and the technical evaluation of contractor's alternative designs with particular reference to foundation design. He has continued his involvement with the project throughout the construction phase. Mr. Farooq specialises in providing expert opinion in relation to bridge design and construction.

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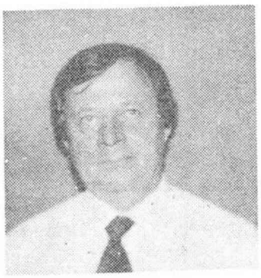

Steve Guest has a wide range of experience in geotechnical consultancy including ground investigation, detailed design, construction and project management. His major civil engineering projects have included: design and construction of heavy foundations, reclamation on very soft clay, flood defence, coast protection and landslide stabilisation.

SUMMARY

This paper briefly outlines the background to the achievement of building a 5km bridge, major river training works and 25km of approach roads to provide the first fixed crossing of the Jamuna River in Bangladesh. A more comprehensive description of the total project can be found in [1]. It then goes on to focus on the design and construction of the foundations. Offshore piling technology was adapted to meet the challenge of constructing piers which could be standing in more than 50m of flowing water. Apart from having to withstand high current forces, they also had to be able to resist boat or barge impact, strong earthquake effects, including 15m of liquefaction of the weak bed material.

Foundations of the Jamuna Bridge – Design and Construction

Background to the Project

Until recently the Jamuna River split Bangladesh in two, separating the agricultural western part of the country from the commercial, industrial and political power bases of the eastern zone; isolating the western zone from potential markets and thereby imposing a developmental stranglehold. However, on 23 June 1998 the first fixed crossing of this mighty river within Bangladesh was opened to road and rail traffic by the Prime Minister Sheikh Hasina, and named the Bangabandhu Bridge in honour of her late father, the founding leader of Bangladesh.

In the flat terrain of Bangladesh the Jamuna's braided channels constantly shift, splitting then reuniting around large sand islands. These sand islands may remain stationary for many years, and become home to many people; then in one flood season they can disappear or move downstream. Below the water, on the bed of the river, sand waves move gradually downstream towards the Bay of Bengal, like desert sand dunes in the wind. Operating ferries on such a capricious river has been difficult, expensive and at times perilous.

Rendel Palmer and Tritton (now High-Point Rendel), together with Nedeco and Bangladesh Consultants Ltd, were initially appointed by the World Bank to carry out studies covering: selection of the preferred crossing site; forecasts of traffic and power demand; alternative forms and configurations of bridges; costs of bridges with and without rail and power, stand-alone power interconnectors, and base case improved ferry services; and through incremental cost/benefit analyses the determination of the best multi-purpose crossing at the optimum location. Additional studies included institutional, environmental impact, resettlement and the planning of townships on flood-free areas created at the ends of the bridge.

The new bridge was required to be 18.5m wide and had to carry a 4-lane roadway, a metre gauge railway, a 230kV electrical power transmission line and a 760mm diameter gas pipeline. It was to be protected from outflanking by major river training works which guide the river under the bridge.

The site chosen for the crossing is some 9 kilometres south of the western bank town of Sirajganj. Here the river is 14km wide in flood, but at low water it can shrink to less than 5km, with changes in river level of 8m. Close to the guide bunds the design scour depth was 47m, and with local scour this increases to moree than 50m.

The contract documents showed due respect to the power and mobility of the river by requiring an unusually flexible approach by would-be contractors. When the contract was signed in May 1994, neither the precise location nor length of the bridge was known with certainty. The position of the eastern end was fixed in October 1994, and of the western end in October 1995.

Due to the variety in contractors' equipment and experience, it was decided to give tenderers the option of bidding on either a design-and-build or a conventional form of contract. Designs for both prestressed concrete and steel/concrete composite decks were prepared, each based on spans of 100m which were found to be reasonably optimal. Additionally, a detailed design specification was included in the tender documents to enable contractors to develop alternative designs.

Tenders were invited in 1993 and the successful tenderer for the main bridge and approach viaducts wa Hyundai Engineering & Construction Joint Venture with a bid of US$247million for a prestressed concret scheme designed by T Y Lin International. The winning tender for the river training works wa US$276million, and the combined approach roads tender was US$56million. A typical bridge module i shown in Figure 1.

The project was jointly financed by the World Bank, the Asian Development Bank, the Japanese OECF and the Government of Bangladesh. The government's contribution came from a fund built up by a special Jamuna Bridge tax levied over nine years prior to start of construction on items such as telephone bills, rail tickets and bank accounts.

The Foundation Challenge

The bed of the Jamuna comprises silty micaceous sands extending to depths of several kilometres. The presence of mica reduces the stiffness and capacity of laterally loaded foundations.

The traditional foundation used throughout the sub-continent has been the open-well caisson. This technique was extended to new limits by Rendel Palmer and Tritton (now High-Point Rendel) between 1979 and 1982 for the first electrical interconnector crossing of the Jamuna River, near Aricha some 60km to the south of the new bridge crossing site.

While this structural form can readily be made strong enough, the loose micaceous soils near the surface allow whole body rotation of the caisson so that it is relatively flexible under horizontal loading. Also, the large diameter causes deep local scour, which adds to the penetration required to ensure stability. For example, the caissons for the first electrical interconnector referred to above were more than 12m in diameter, and allowance for local scour below regime scour was some 24m. These caissons were sunk with the help of a bentonite annulus to more than 100m into the bed of the river; but even then under maximum scour a lateral displacement of 1.2m was estimated as the maximum at the top of the caisson. Fortunately, power lines are relatively insensitive to lateral displacements. Early in the bridge study analyses were carried out on single and multiple groups of vertical founding elements and in every case predicted deflections were found to be excessive.

In addition to the inadequate lateral stiffness, two other concerns confirmed the study team in the opinion that caissons would not provide suitable foundations for the Jamuna Bridge. Firstly, there were doubts surrounding the feasibility of constructing a scheme with caisson foundations within the timescale required, even if all work progressed without incident. Secondly, there was the question of the risk associated with their installation. Of the 11 caissons for the first electrical interconnector there were serious problems with two of them.

Foundation Design

Design is inextricably linked with the construction method for major bridges, and never more so than in the case of the Jamuna Bridge. Having questioned the viability of caissons a new approach was required. In the 10 years or so prior to our Jamuna work there had been dramatic developments in over-water piling capability in response to the oil industry's need to install offshore platforms in deeper and deeper water. The large floating plant and piling hammers have proved their reliability driving large diameter steel piles in both fiercely hostile ocean environments and in the calmer environment of rivers or estuaries. When the project was being planned the offshore construction industry was somewhat depressed, and it seemed that there was an opportunity to take advantage of this foundation installation technique which was both reliable and fast.

So it was that we turned to large diameter raking steel piles. By resisting horizontal loading through axial load, a raking pile system is stiffer than caissons or vertical piles. Resistance is provided by skin friction and end bearing at depth, in soil which is strengthened simply by the weight of surcharge. To give some idea of the difference in lateral stiffness, a 100m deep caisson of 7.5m diameter was found to be more than six times as flexible as a raking pile foundation with pile toe levels some 20m higher than the caisson founding level.

Hence a raking pile foundation was found to be a more efficient structural system, and has the added virtue of being more transparent to the river flow. However, given the 50m design height from bed and a river flowing at more than 4m/sec, vibrations induced by vortex shedding in the stream flow could cause potentially disastrous oscillations in the piled structure or in individual piles. Calculations indicated that the piles would need to be at least 2.5m in diameter to avoid this problem.

High-Point Rendel prepared alternative reference designs with both steel and concrete decks to encourage competition, but both schemes were supported on raking steel cylinder piles. Raking precast concrete cylinder piles were considered non-competitive due to the difficulty of installation. Groups of 2 and 4No 2.5m diameter piles of maximum wall thickness 50mm worked well with our steel/concrete composite superstructure, and groups of 4No 2.5m diameter of similar wall thickness were used for the concrete scheme. Hyundai's concrete scheme was supported on groups of 3No 2.5m diameter and 2No 3.15m diameter piles, with wall thicknesses that varied between 40mm and 60mm. The arrangement of these piers is shown in Figure 2.

Geotechnical Investigations

Pre-Contract Investigations

The site lies in the Bengal geosyncline which is continually subsiding, leading to the deposition of sediments brought down from the upper reaches. At Sirajganj the depth to basement rock is as much as 6km.

Soil investigations undertaken between 1986 and 1988 during Phases I and II of the Feasibility Studies approximately 1km from the final alignment showed recent alluvial silty sands, loose at the surface becoming medium dense with gravelly layers below a depth of about 50m extending to about 100m where hard silty clay overlies a dense mica silt.

The investigations by the Japanese International Cooperation Agency in the 1970's a few kilometres to the south of the 1986 and 1987 tests undertaken during our study indicated a similar sequence but with gravel encountered at about 80m overlying Pleistocene sands and gravels.

For foundation engineering design the soil characteristics of importance were the density, the angle of shearing resistance and compressibility. Given the nature of the soil it was decided that these could best be determined by means of established relationships with the results of static cone penetration tests.

The data obtained from the CPT tests were correlated by visual examination with soil samples obtained from boreholes, and with the results of laboratory classification. Values of cone resistance and sleeve friction did not vary widely from one location to another across the river. This made it possible to establish what were believed to be representative relationships for the whole crossing between average and lower bound cone resistance with depth below bank or mid-river char level.

It was necessary to predict the influence on soil properties of deep scour and subsequent redeposition, and also the likely behaviour of the soil under earthquake shaking. The unloading effects due to scour were analysed by relating the observed q_c/depth records to the reduction in horizontal stress caused by scour. Through granular interlock the soil mass retains some proportion of the horizontal stresses from previous loadings. A theoretical relationship was established to determine this remaining horizontal stress as a function of the degree of unloading. Curves were thus established to provide an adjustment factor to be applied for various bed levels, enabling the design cone resistance vs depth relationship to be calculated. An overwater CPT test was carried out in the deepest nearby channel for comparison with predicted values from this scour unloading theory.

The sandy deposits were found to be considerably more compressible than typical normally consolidated quartz sands, partly perhaps as a result of their significant mica content.

Investigations during Construction

A comprehensive soil investigation (including piezocone, seismic cone, cone pressuremeter, boreholes and standard penetration tests) was carried out as part of the main contract prior to construction, and instrumented reduced scale pile tests on three 760mm diameter steel piles were performed to determine the actual skin friction. Whereas the driving resistance was as anticipated, the results of these reduced scale tests indicated that previous assumptions regarding skin friction were over-optimistic. However, a pull-out test carried out 9 months later on the same reduced scale pile showed that shaft friction had increased by a factor of 2.7. Cone penetration tests at each pier location confirmed the ground conditions assumed in design, and dynamic pile testing at selected piers (see below) verified the pile capacity predicted by geotechnical theory.

Base resistance constituted the major component of resistance of pile capacity for deep scour conditions, with or without earthquake liquefaction. The overall pile length required little adjustment because the presence of a sandy gravel layer at around -70mPWD compensated for the apparently low skin friction.

Site-Specific Seismicity Study

The site is within an area of significant seismic activity. Reports of an earthquake in 1885 suggested a magnitude of about 7 with its epicentre approximately 50km from the site. It is probable that this event emanated from the Bogra fault system which lies some 25-50km to the NW. The site is also subject to more moderate shaking from more distant seismic zones in the foothills of the Himalayas, to the N and NE. However, the most important effects stem from the near-field events in the Bogra fault system.

A site specific seismology study was carried out which recommended a design stiff soil peak acceleration of 25%g corresponding to 1-in-100year ground motions generated at a source distance of 25km by a magnitude 7 earthquake. Shear wave velocity tests at site using a seismic cone indicated a depth to "engineering stiff soil" of between 300 and 500m. The stiff soil peak acceleration was translated to a peak acceleration close to the surface of 20%g. Imperial College supplied six representative earthquake records from their strong motion data bank, and these were used as input for the detailed soil/pile/structure interaction analysis.

Correlation of the field and laboratory tests with published information indicated that the soils were potentially liquefiable to a depth of 15m below river bed level under earthquake shaking. Computer analyses using representative time histories on a model simulating pore pressure generation and dissipation confirmed 15m of liquefaction was a suitable design value. The relationship between pore pressure and depth below the liquefied zone was also established, and the soil resistance in this transitional zone was adjusted accordingly in the earthquake load case.

Damage to the foundations as a result of a severe earthquake would be difficult to repair. Limited local seismic data, an infinitely variable bed profile, and liquefiable soil reduced the reliability of response prediction. Also, it was difficult to generate consistent ductility in the pier stems, which varied from more than 12m high at the middle of the bridge to less than 3m high at the ends, to safeguard the foundations from excessive load in an extreme seismic event. Protection was therefore provided by elasto-plastic load limiting devices between the deck and piers at bearing level.

Foundation Construction

The bridge site lies some 300km upriver from the Bay of Bengal. At the site, before constriction of the river by the training bunds, at low water only about half the 4.8km bridge length was over water and the rest was over flood plain and sand islands. To work with floating plant a contractor would have to dredge and maintain a channel along the line of the bridge to provide the 4.5 to 5m draught needed for the piling barge. The High-Point Rendel construction planning study in 1988 indicated that the piles for the whole bridge could be installed within a single season, and the piling barge could get to and from the site without extensive dredging during the flood season. This was important because to have the expensive barge and hammer sitting idle during the monsoon season, or unable to demobilise from site would incur high additional costs.

The contractor fabricated the piles at Ulsan in South Korea from plate purchased mainly from Japan. First, 4m "cans" were rolled and seam welded, then these were welded together to make up the lengths for shipment and installation. The majority of the piles were to be installed in two lengths, with the top section being fully butt welded after driving the first. The longer piles near the guide bunds were installed in three sections.

The HD-1000 pile driving barge with the new Menck MHU 1700T hydraulic hammer arrived on site in September 1995. Its first task was to drive two full size (3.15m diameter) trial piles near the eastern end of the bridge. These piles were to demonstrate the installation methods, including driving, welding, clean out to within 3 diameters of the toe, concreting, and pressure grouting at base of concrete.

On 15th October 1995 the position of the west guide bund and, therefore, the west end of the bridge was determined. This allowed the bridge alignment and the location of each of the piers to be fixed. The sequence of pier construction was (see Figure 3):

- Position piling template (or "jig jacket") on river bed and fix with temporary piles;
- Lift first pile length into jig and lower to soil;
- Drive first pile length, and cut off top section of pile with lifting holes and possible hammer damage;

[Cutting was done by a carriage-mounted acetylene/oxygen flame torch guided by a strap fixed around the pile circumference. The torch was angled to produce a bevel of 15° and the cut surface was ground smooth ready to receive the next length of pile which had been prepared to a 30 ° bevel. Shims welded to the face of the lower pile ensured constant root gap along the weld].

- Pitch upper pile section and butt weld the two lengths using a manual submerged arc process, drive to required toe level, and cut off top section (see Figure 4);

  [The weld and adjoining pile wall was ultrasonically tested. During the driving process, a proportion of piles were monitored by dynamic pile testing, and analysed using the CAPWAP method].
- Drive other piles in pier;
- Clean out piles down to two diameters above toe level using an airlift (see Figure 5);

  [Air under pressure was pumped down to provide the cutting and lifting action: A head of water between 0 and 5m above river level was maintained with submersible pumps].
- Install grout pipe cages and pour infill tremie concrete in pier piles (see Figures 6 and 7);

  [The grouting system comprised tubes-a-manchettes. The piles were filled with concrete to provide reliable end bearing and ductility. When the pile infill concrete had begun to set, but before it had hardened, water was pumped under pressure to crack the concrete surrounding the manchettes. The piles were subsequently grouted to pressures of 60 and 50 bars for the 2.5m diameter and 3.15 m respectively. The pressures were selected to restore the stress in the disturbed soil at the pile toe level, without risking pile uplift during grouting].
- Pressure grout at base of concrete infill;

  [When the concrete had hardened sufficiently, grout was pumped to the soil/concrete interface under 40 bar pressure. The purpose of base grouting was to reinstate the stiffness of the soil loosened by pile clean-out. CPTs through the concrete plug of the trial piles showed that only about 0.7m depth was loosened. Below that, the soil plug remained denser than the undisturbed soil. Base grouting was an insurance policy against gross loosening. It was seen as a way to keep absolute and differential settlements to a consistent minimum. Measured settlements to date have been roughly in accord with predictions].
- Install precast concrete pilecap shell;

  [The precast concrete shells weighing up to 300 tonnes were loaded out and lifted into position using a barge mounted crane. The shell was temporarily supported from the piles whilst the concrete plug was cast under water. The shell was then de-watered, reinforcement was fixed and the structural concrete cap was poured. Photograph ? shows a typical pile cap shell being lowered over the piles].
- Cast variable height pier stems to bearing level.

Conclusion

Innovative application of offshore piling technology made possible this crossing 300km upriver from the Bay of Bengal. The two full-size trial piles and all the 121 permanent works piles were successfully driven between October 1995 and June 1996, and the barge was sailed back down river with two months to spare before shallow water would have stranded it until the following flood season.

Acknowledgements

The authors would like to thank Dr A K Abdul Mubin, of the Jamuna Bridge Division of the Bangladesh Ministry of Communications for his kind permission to publish this paper. Thanks are also due to Hyundai Engineering and Construction JV for making photographs available; and last but not least to Mr Michael Tomlinson who was an inspirational member of the team.

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JAMUNA MULTI-PURPOSE BRIDGE
BANGLADESH

High-Point Rendel
capital management – engineering certainty

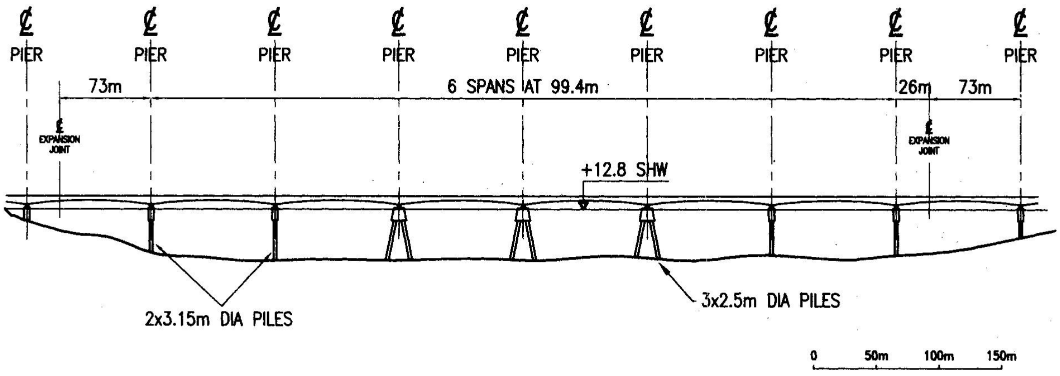


Figure 1 Elevation on 7-Span Module (overall length of main bridge 4.8km)

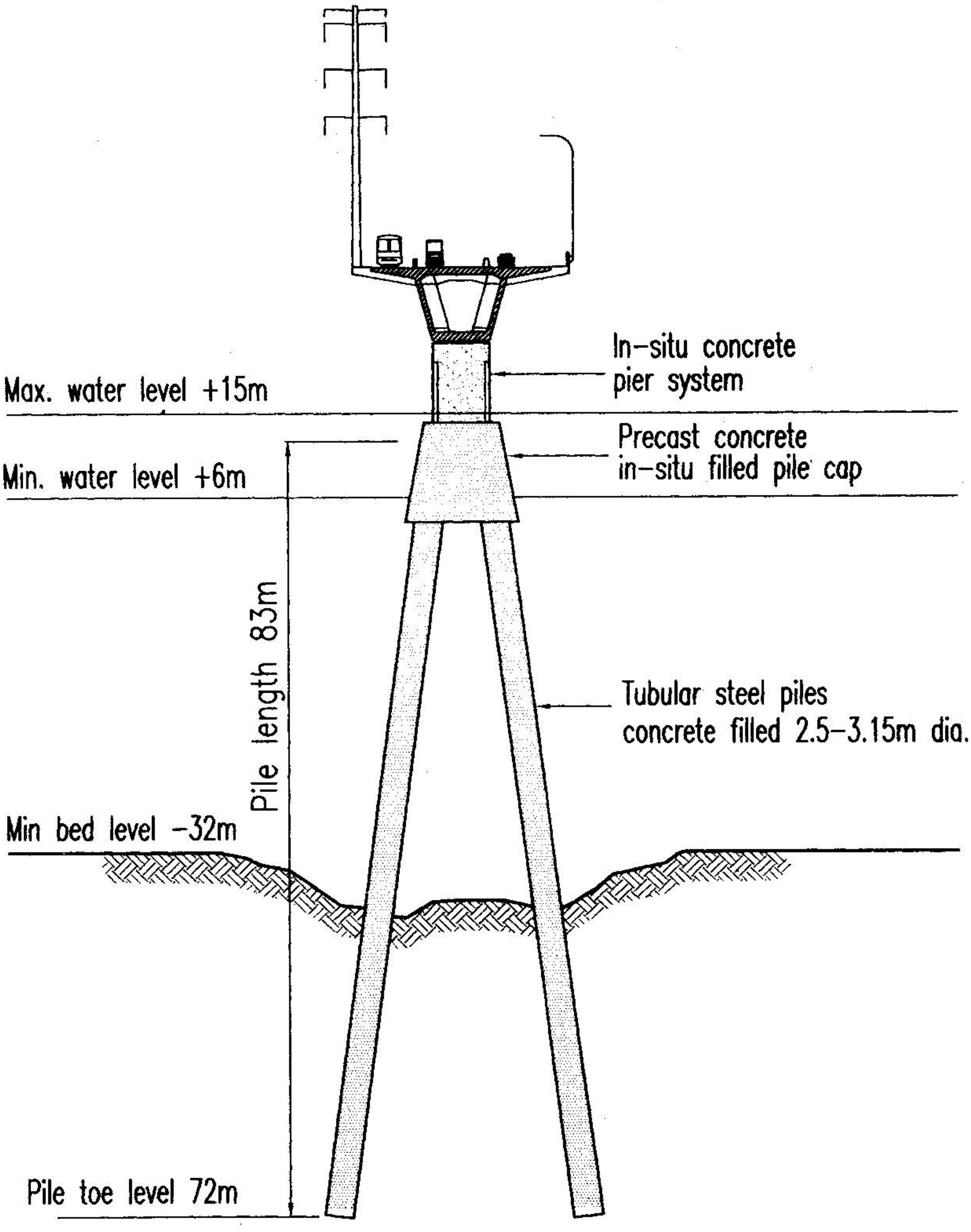


Figure 2 Bridge Section showing Piled Foundations

JAMUNA MULTI-PURPOSE BRIDGE BANGLADESH	High-Point Rendel **High-Point Rendel** *capital management – engineering certainty*

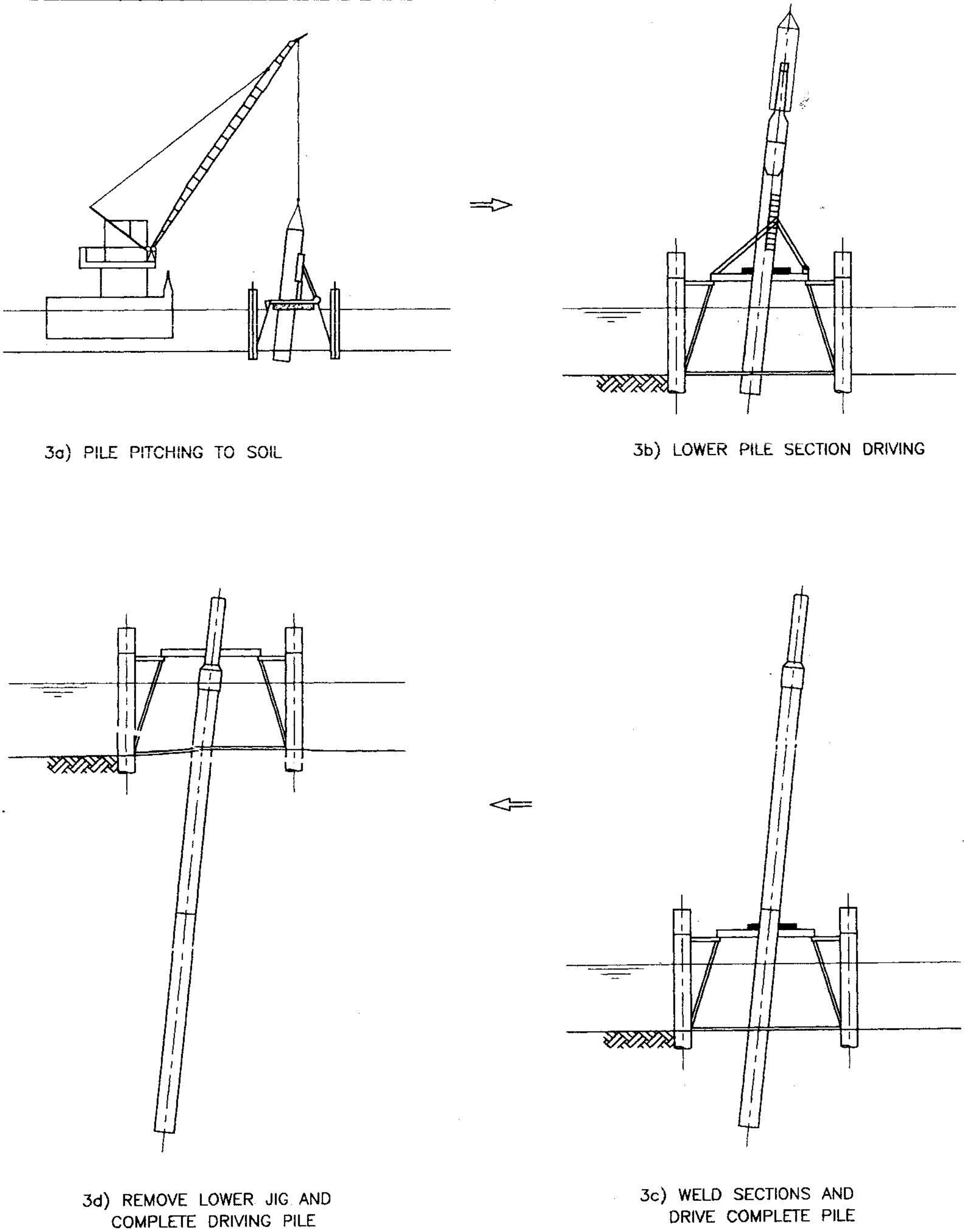


Figure 3 Pile Installation Sequence

JAMUNA MULTI-PURPOSE BRIDGE BANGLADESH	**High-Point Rendel** capital management – engineering certainty

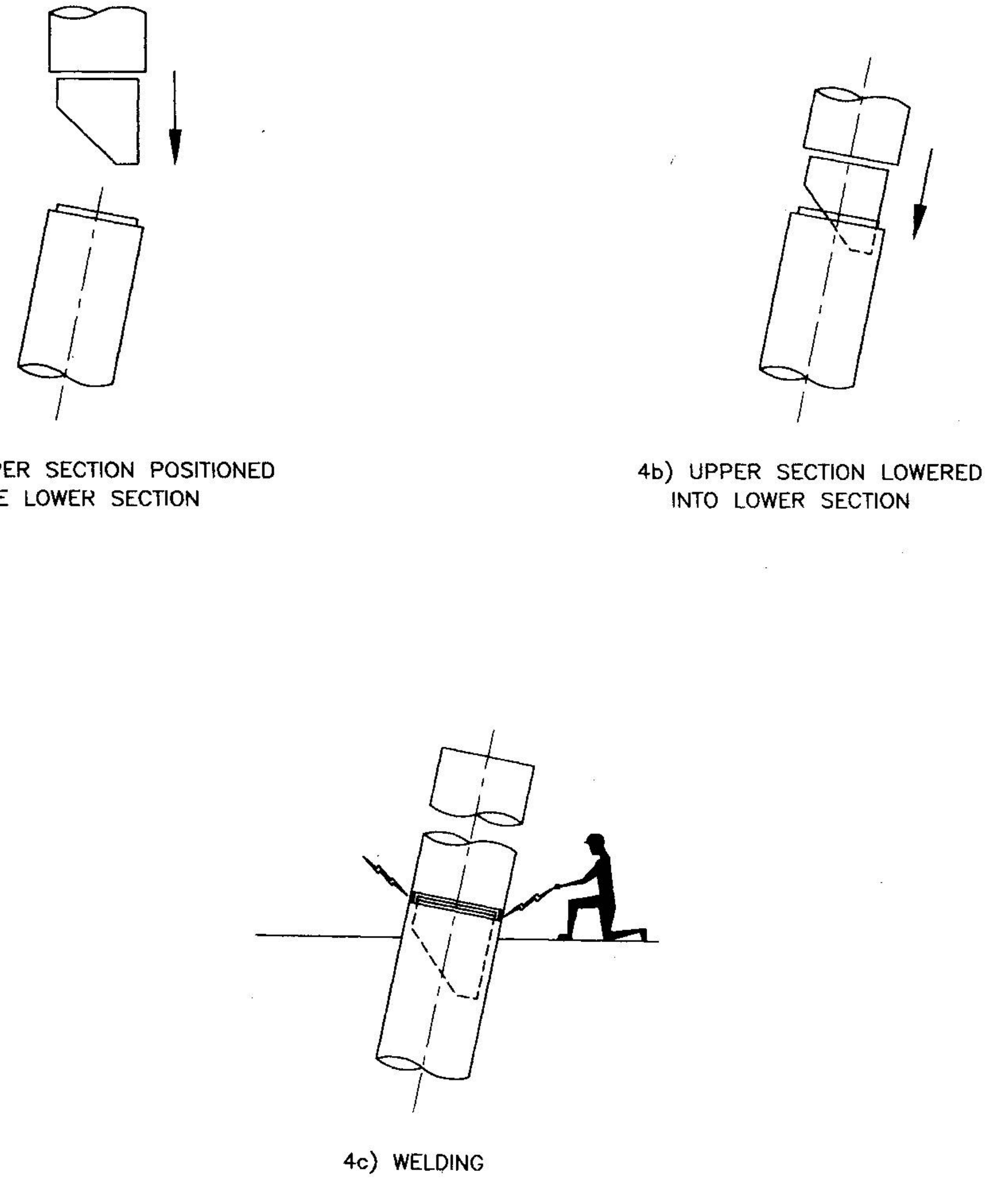


Figure 4 Positioning and Welding Pile Splices

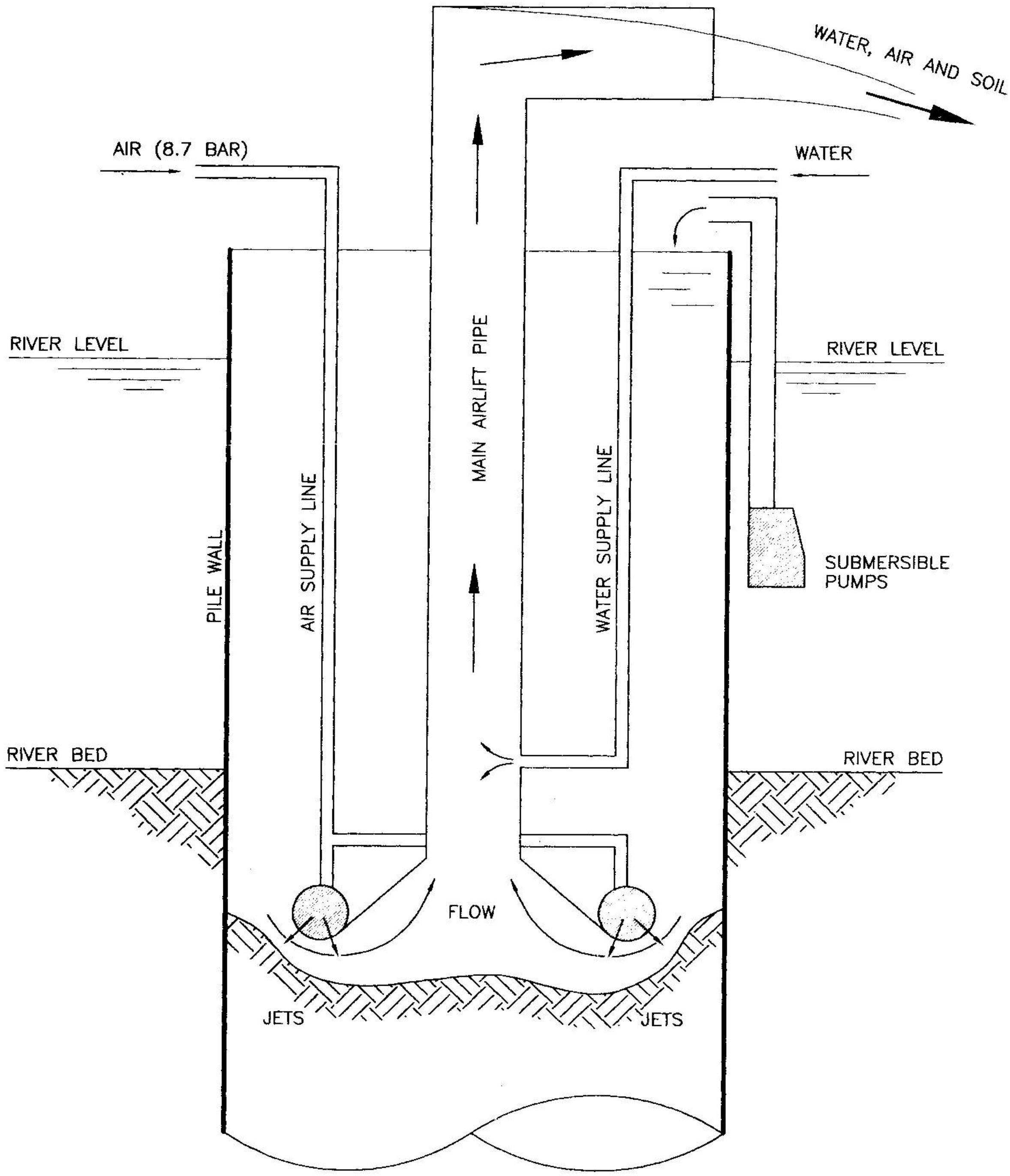


Figure 5 Removing Soil from Pile Using Airlift (Diagammatic)

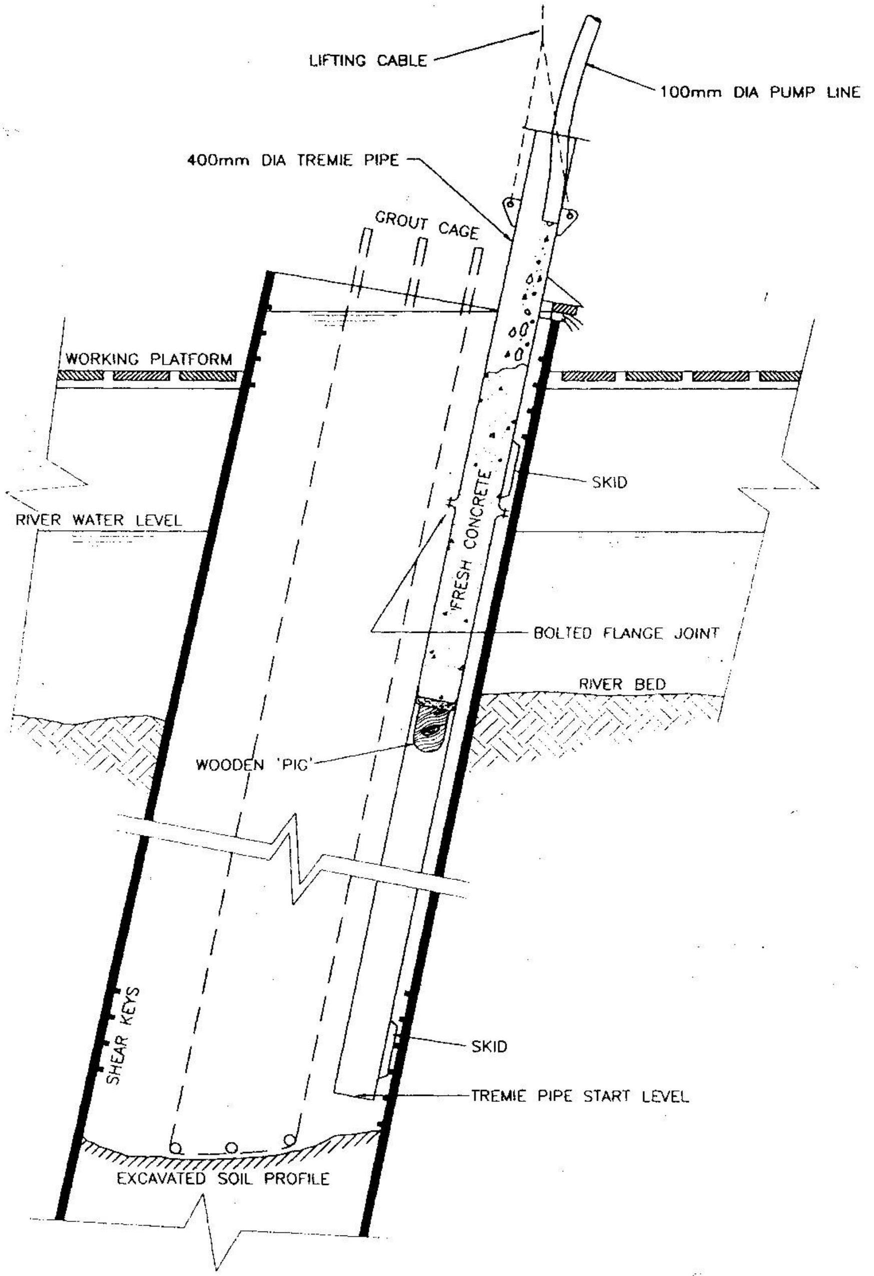
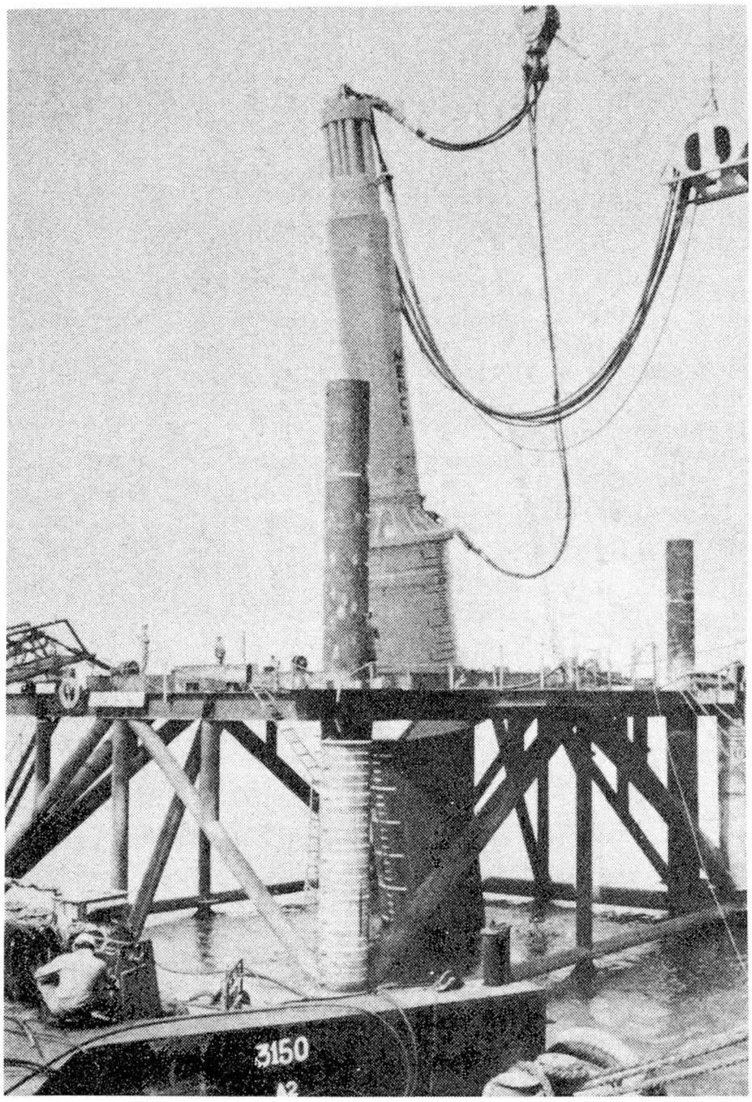


Figure 6 Tremie Pipe Setting and Use

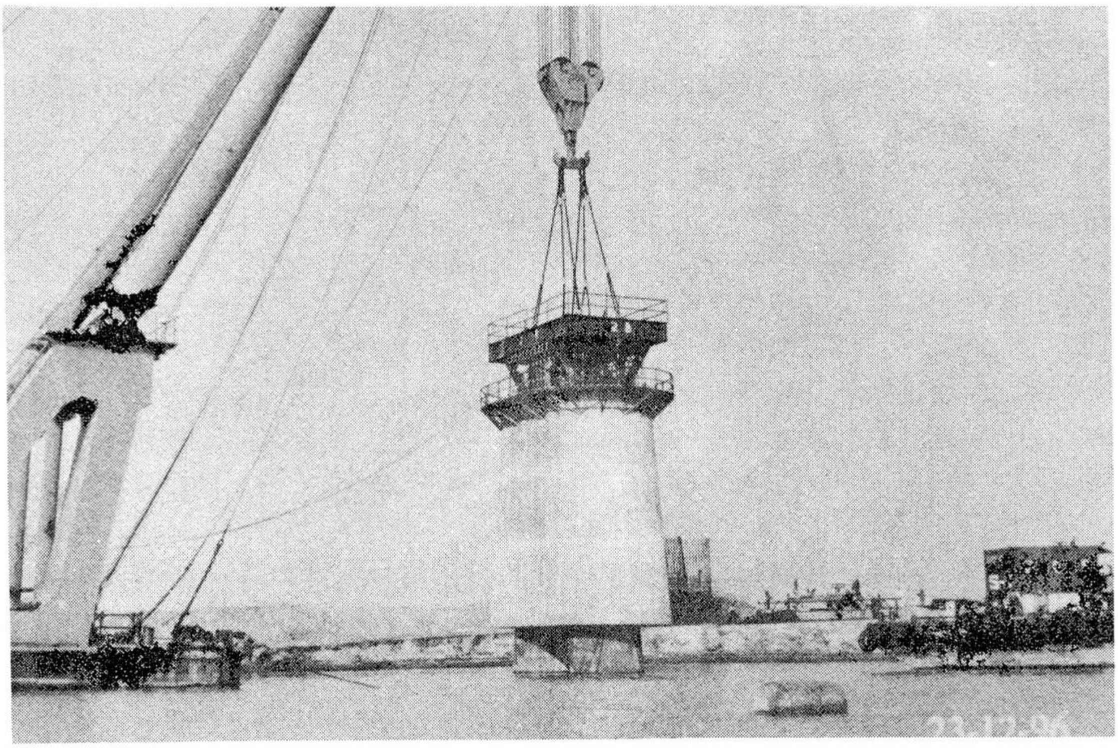


JAMUNA MULTI-PURPOSE BRIDGE

High-Point Rendel

Photo 1 Driving a 3.15m Diameter Pile using the Menck 1700 Hammer (left)

Photo 2 Lifting a precast pilecap shell into place (below)



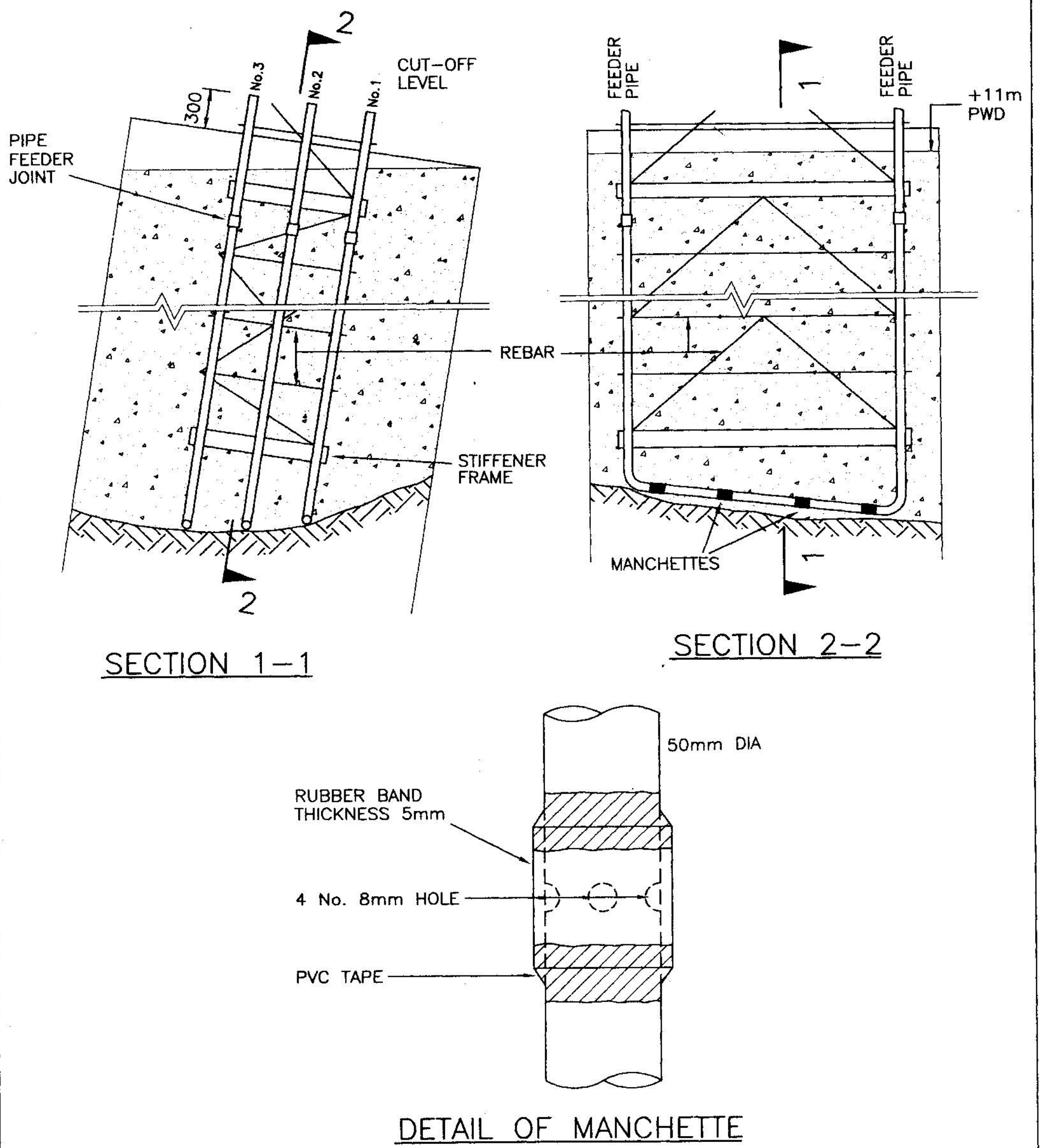


Figure 7 Pressure Grouting using Tube-a-Manchette Technique

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