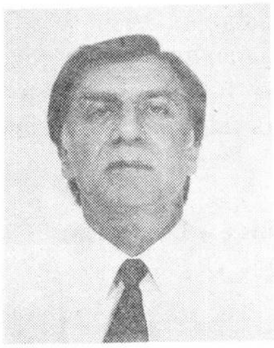
Foundations of the Jamuna Bridge: design and construction

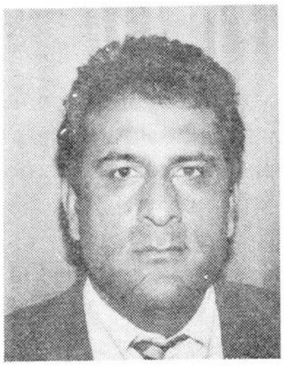
Foundations of the Jamuna Bridge – Design and Construction

Joe M BARR
MSc, DIC, CEng, MICE, MASCE
Technical Director
Head of Concrete Bridge Engineering,
High-Point Rendel
London, UK



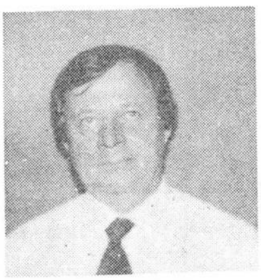
As Chief Bridge Engineer, Joe Barr has been closely involved with the Jamuna Bridge project from the early studies through construction. His areas of specialist expertise include segmental concrete bridges, caisson and piled foundations, seismic engineering, risk and reliability analysis, procurement 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.

Abdul FAROOQ
B.Sc(Eng), CEng, MICE, ACIArb
Technical Director
Head of Project and Technical Audit
High-Point Rendel
London, UK



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.

Steve GUEST
MSc, CEng, MICE,
Technical Director
High-Point Rendel
London, UK



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.

Reference

1. Tappin R G R, van Duivendijk J, Haque M, The Design and Construction of Jamuna Bridge, Bangladesh. Proceedings of the Institution of Civil Engineers, November 1998.

JAMUNA MULTI-PURPOSE BRIDGE
BANGLADESH

High-Point Rendel
capital management – engineering certainty

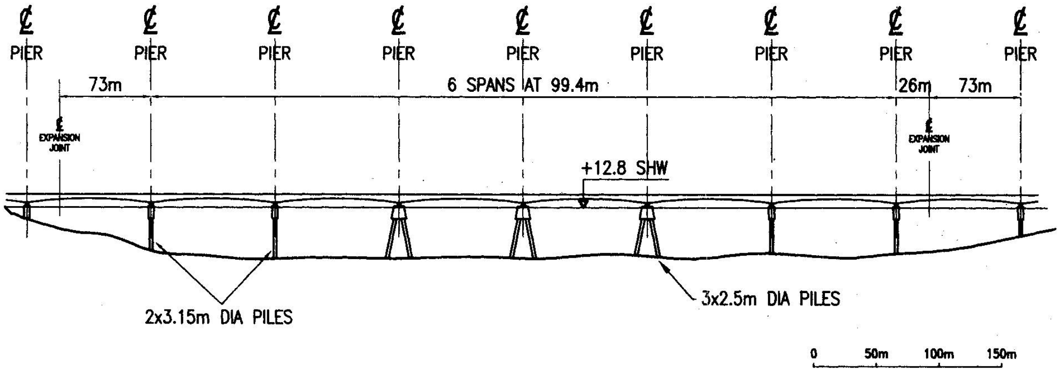


Figure 1 Elevation on 7-Span Module (overall length of main bridge 4.8km)

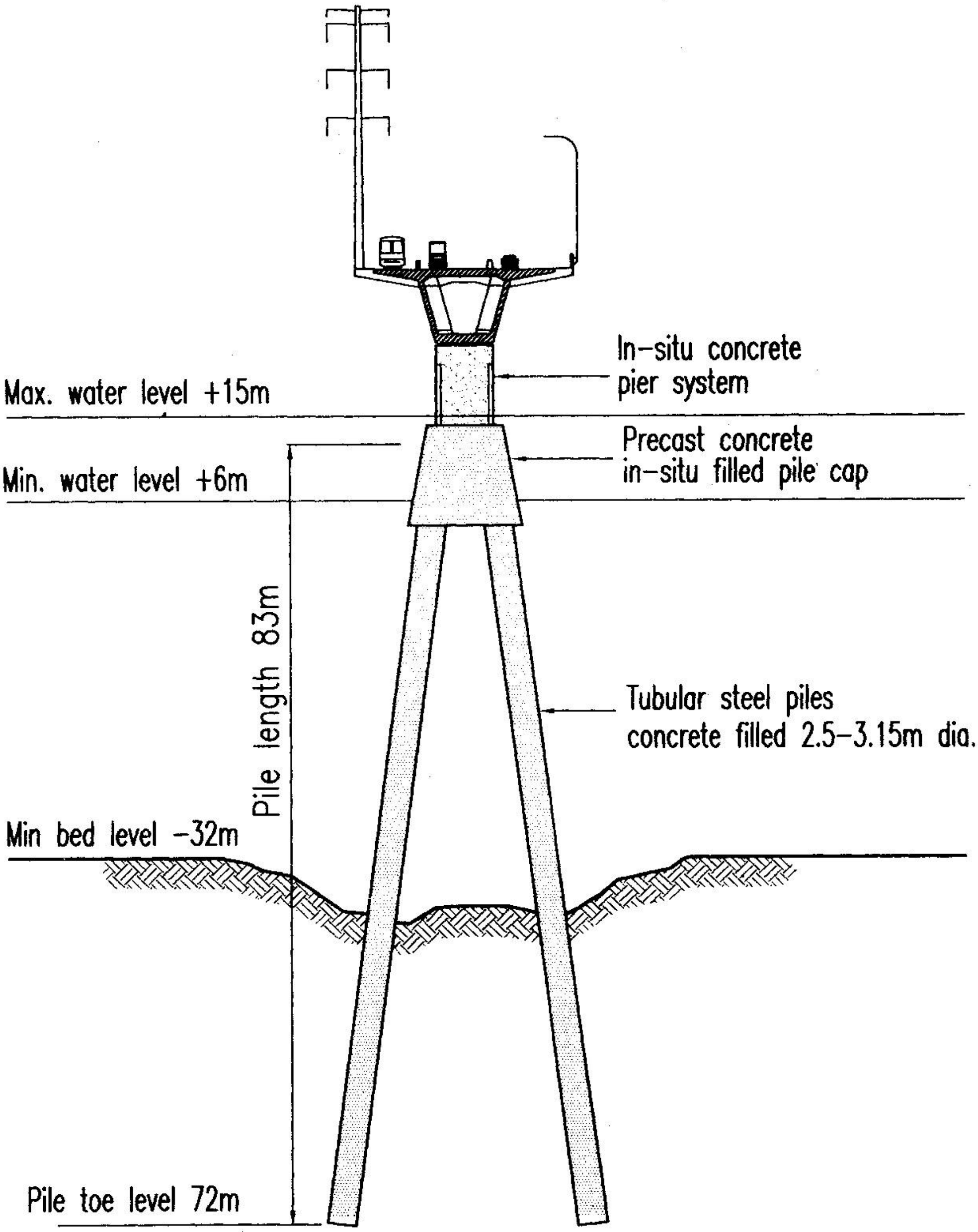


Figure 2 Bridge Section showing Piled Foundations

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

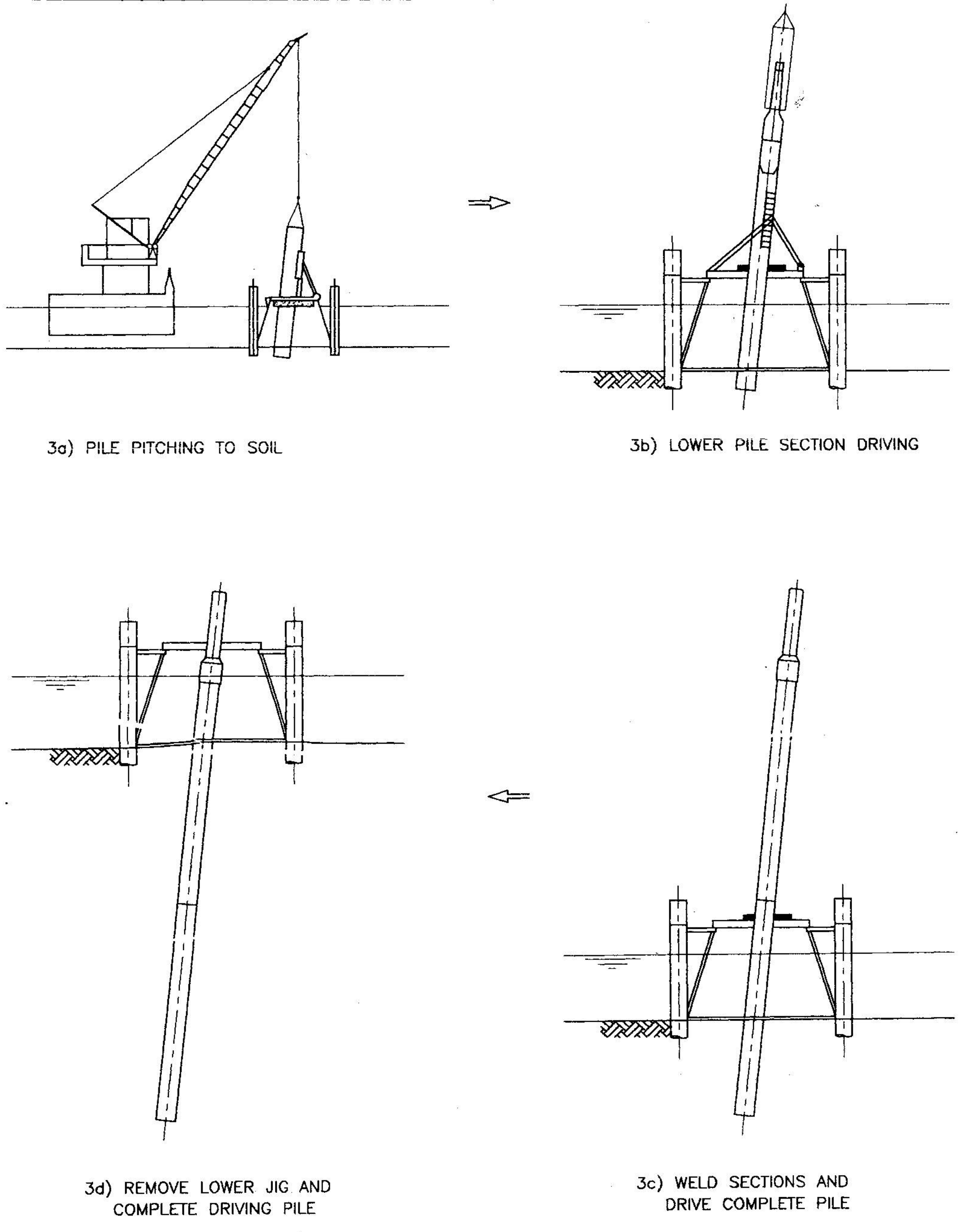


Figure 3 Pile Installation Sequence

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

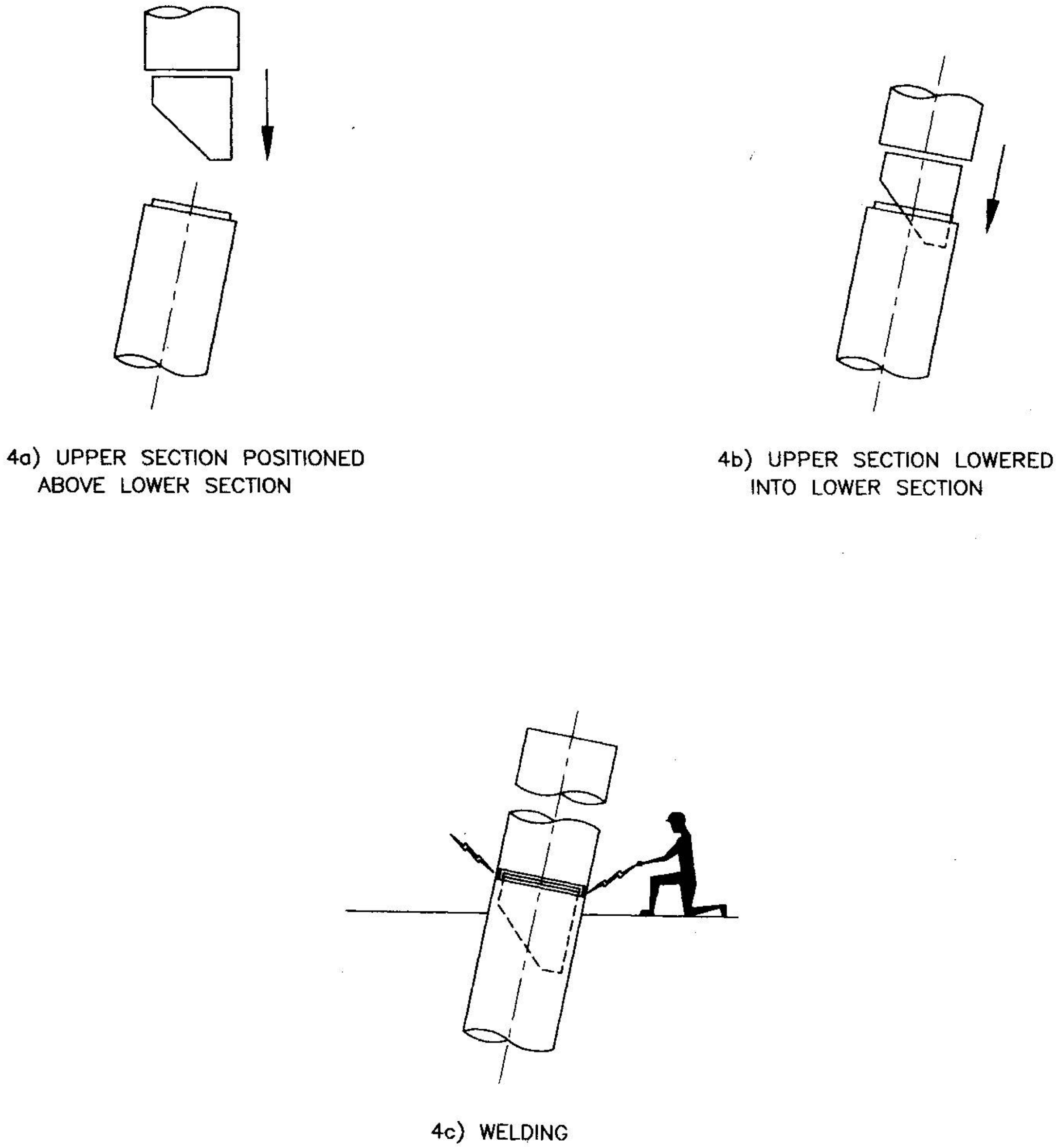


Figure 4 Positioning and Welding Pile Splices

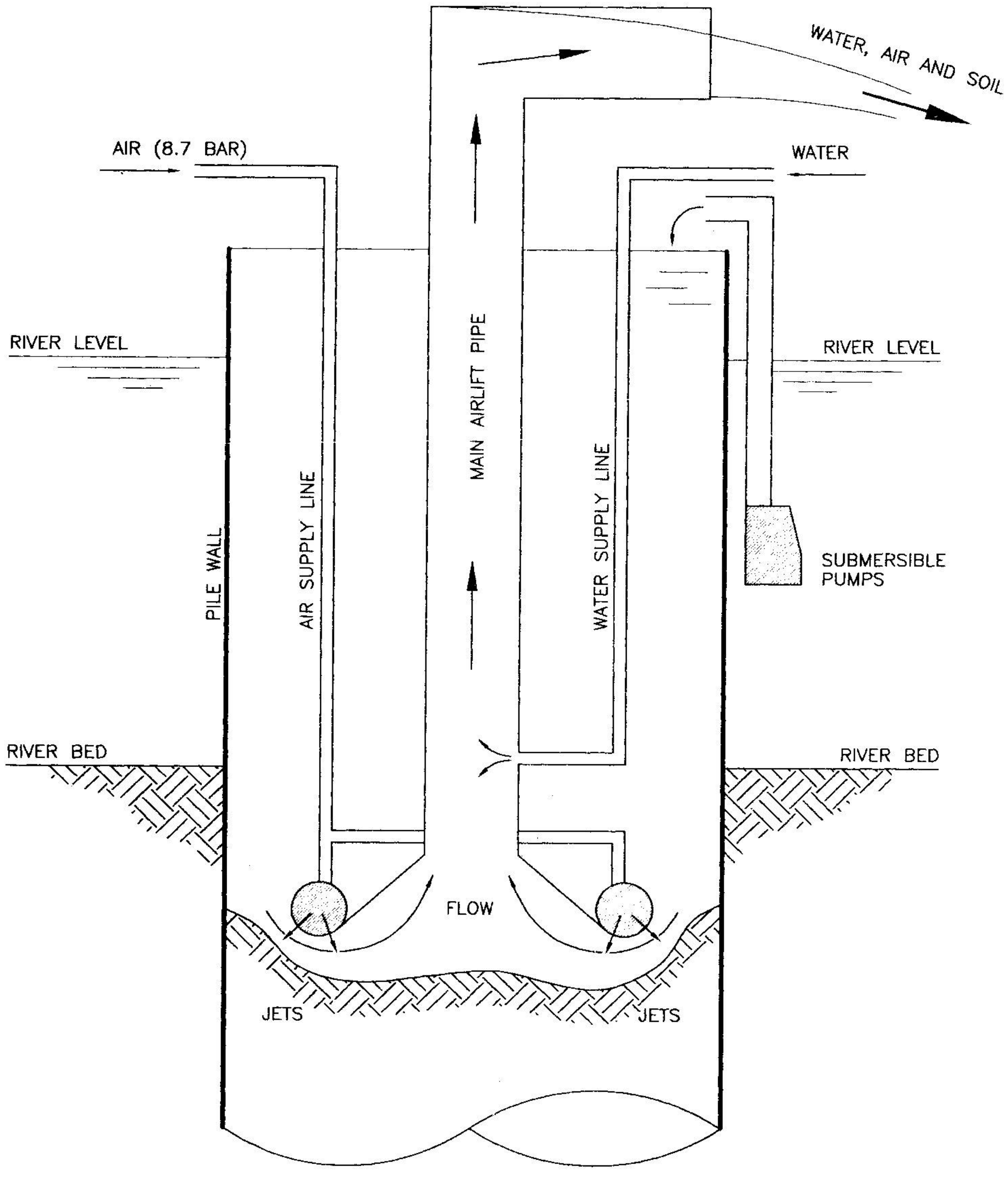


Figure 5 Removing Soil from Pile Using Airlift (Diagammatic)

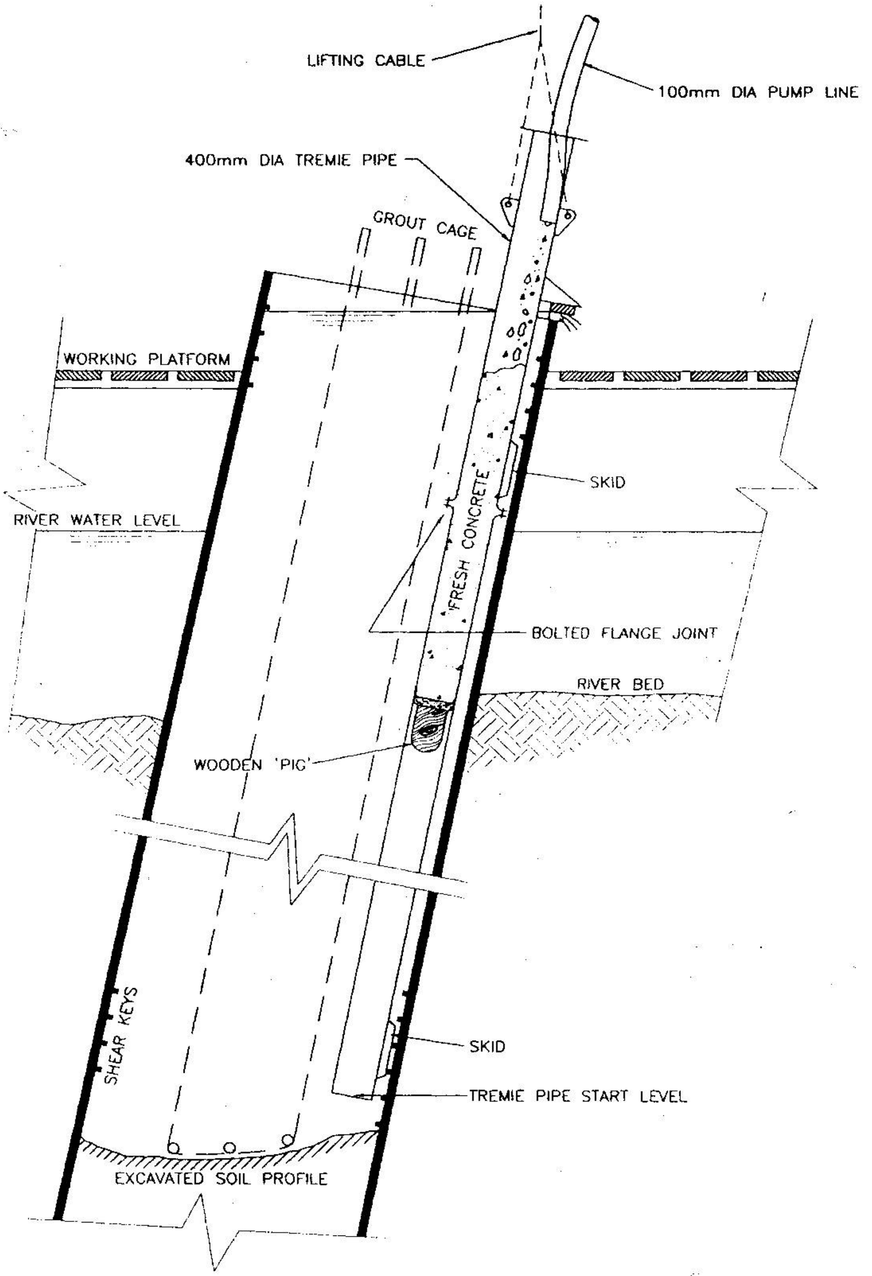
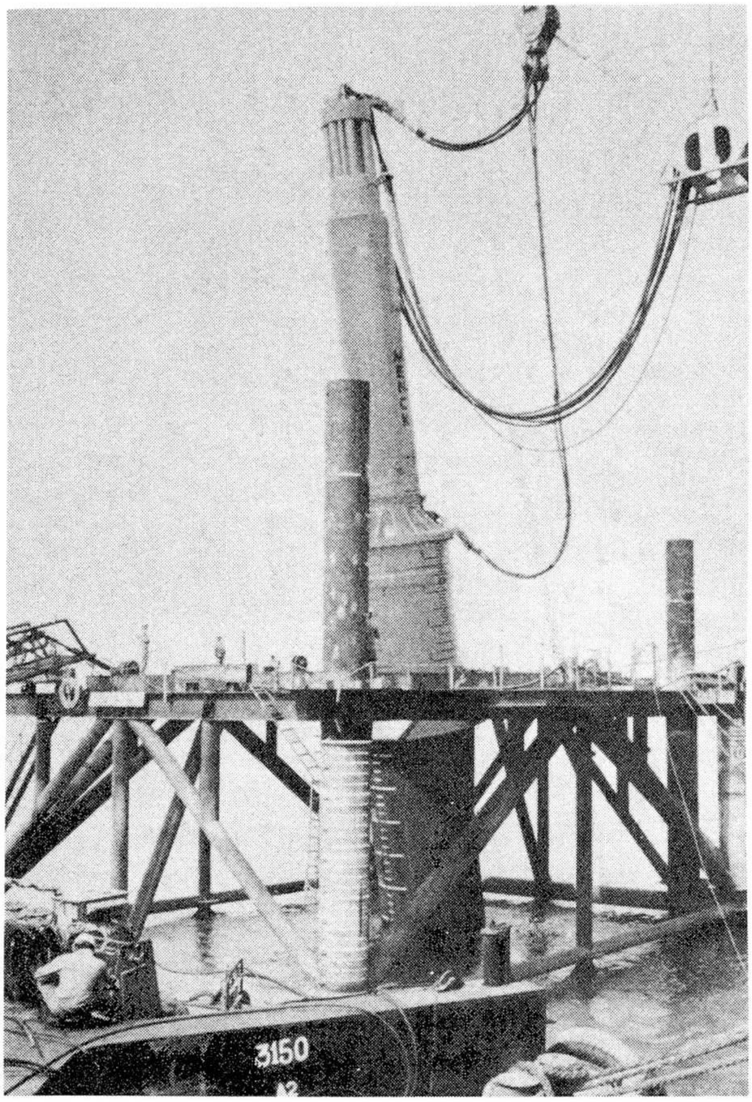


Figure 6 Tremie Pipe Setting and Use

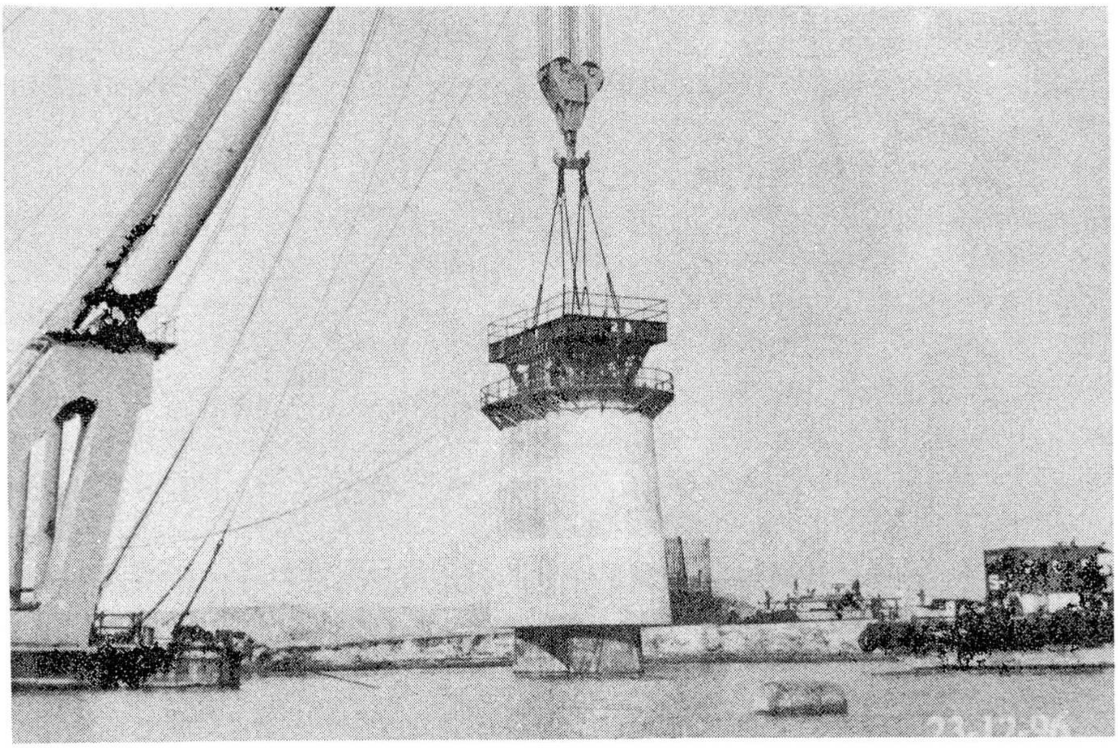


JAMUNA MULTI-PURPOSE BRIDGE

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)



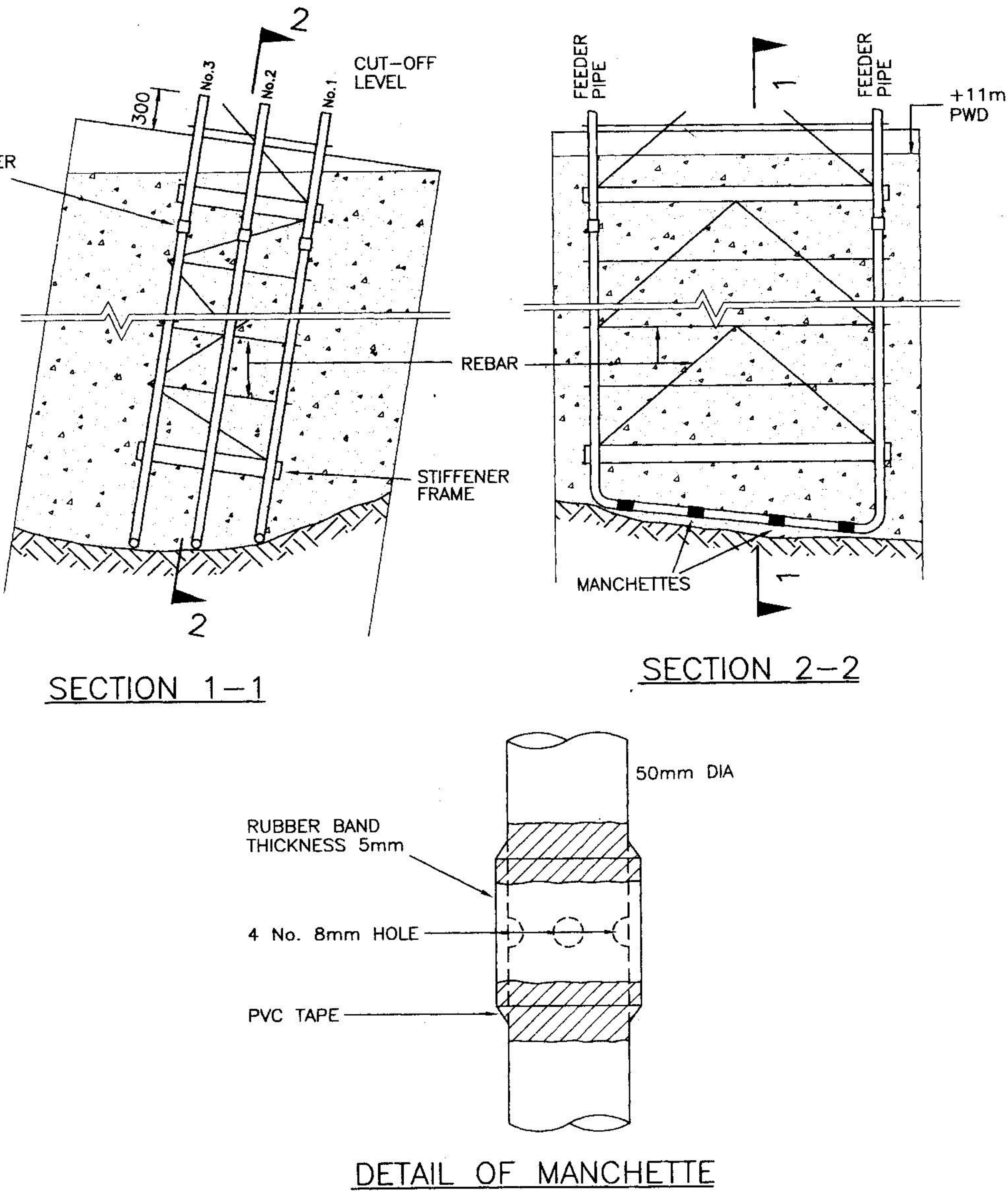


Figure 7 Pressure Grouting using Tube-a-Manchette Technique

Leere Seite
Blank page
Page vide