Effect of Dredging and Axial Load on a Berthing Structure

K. Muthukkumaran, *Lecturer, Department of Civil Engineering, National Institute of Technology, Tiruchirappali - 620 015, Tamil Nadu, India, Tel.: +91 431 2501801; Fax: +91 431 2500133, Email: kmk@nitt.edu*
R. Sundaravadivelu, *Professor & Head, Department of Ocean Engineering, Indian Institute of Technology Madras, Chennai-600 036, India, Tel.: +91 44 2257 4801; Fax: +91 44 2257 4801, Email: rsun@iitm.ac.in*
S.R. Gandhi, *Professor, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India, Tel: +91 44 2257 4255; Fax: +91 44 2257 4252, Email: srgandhi@iitm.ac.in*

ABSTRACT: *Piles and a diaphragm wall of a supported berthing structure on marine soils are loaded both axially and laterally. Axial loads are typically generated by the self weight of the structure and external live loads; lateral loads are typically generated by wave current and seismic loads. These loads are generally considered in the design of berthing structures. However, the lateral force generated by lateral soil movements due to dredging may not be considered or accounted in the design of berthing structures. Hence, a full-scale field study was conducted on a berthing structure to estimate the actual axial load distribution during axial loading and the lateral displacement during dredging. After construction of the berth, it was decided to conduct a full scale axial load test on a single pile and monitor the lateral movements of the berth during and after dredging. This paper describes the load transfer data obtained from a bored cast – in – situ pile socketed 1.5 m into hard rock and the lateral displacement of the berth recorded during and after dredging to a -9.5 m dredge depth.*

KEYWORDS: Berthing Structures, Dredging, Lateral Soil Movement, Socketed Pile, Consolidation

SITE LOCATION: IJGCH Geographic Database

INTRODUCTION

A shallow water berth at Jawaharlal Nehru Port Trust (JNPT), Mumbai was recently constructed and comprised of a diaphragm wall and pile rows to support the deck structure. The JNPT is one of the major ports in India. The soil strata consisted essentially of very soft clays extending to large depths. Reclaiming land behind this structure created a number of problems including lateral soil movements. The structure was constructed following placement of fill necessary for reclamation.

The load transfer mechanism from a pile to the founding strata is a complex interaction problem involving the different rigidities of various soil strata with respect to the pile. Simplified theoretical solutions do not always provide a sound basis for the assessment of load transfer in different layers; hence, pile instrumentation tests are often performed to measure the axial load at various elevations. After completion of the structure, the instrumented pile load test was conducted on a pile with a design load of 9000 kN. The compression test was carried out in an operational rock-socketed pile which showed excellent behavior. The results have been reported previously elsewhere (Das, 1983). The instrumented pile load test has been performed in the past with varying degrees of success. Whitaker and Cooke (1966), Barker and Reese (1969), Williams (1980) and Gandhi et al. (1987) suggested different types of load sensors to determine the vertical and radial stress acting on a pile. Young and Chin (1981) presented practical aspects of the instrumentation related to installation and performance. The authors provided failure rate for different instrumentation systems and recommended redundancy in the instrumentation program since damage or malfunctioning of instruments is unavoidable during pile installation.

The development of port structure necessitated in-depth studies on the behavior of berthing structures during dredging. Most of the coastal regions have sloping seabeds with low shear strength, soft, marine clay. Therefore, slope stability is a common issue in these areas and the potentially unstable slopes may create problems to existing structures. The clay strata

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may cause lateral movements and transfer additional large lateral forces to the pile causing damage. When the current dredging work was undertaken, it was decided to monitor the lateral movements of the berth. For this purpose, one inclinometer tube was installed in one panel of the diaphragm wall and in one of the piles of the structure.

The magnitude of the soil movement is related to many factors such as soil properties, structural properties and dredging sequence. A number of case histories have been reported in the literature and have investigated the relationship between those factors and wall deflection. The effects of diaphragm wall construction and the additional ground response have been studied for soil conditions in the United States, Europe and Asia. Among these are Dibiagio and Myrvoll (1972), Farmer and Attewall (1973), Davies (1982), Tedd et al. (1984), Clough and Rourke (1990) and Tamano et al. (1996). These studies investigated, in some cases using numerical models, the effect of wall construction on ground movements and changes in lateral earth and water pressure. For Singapore soil conditions, Chen and Yap (1991) reported the effects of the construction of a diaphragm wall panel on an adjacent old masonry building. Poh et al. (2001) presented the effects of a diaphragm wall construction in Singapore soils. However, field data on the lateral soil movements during dredging are still limited.

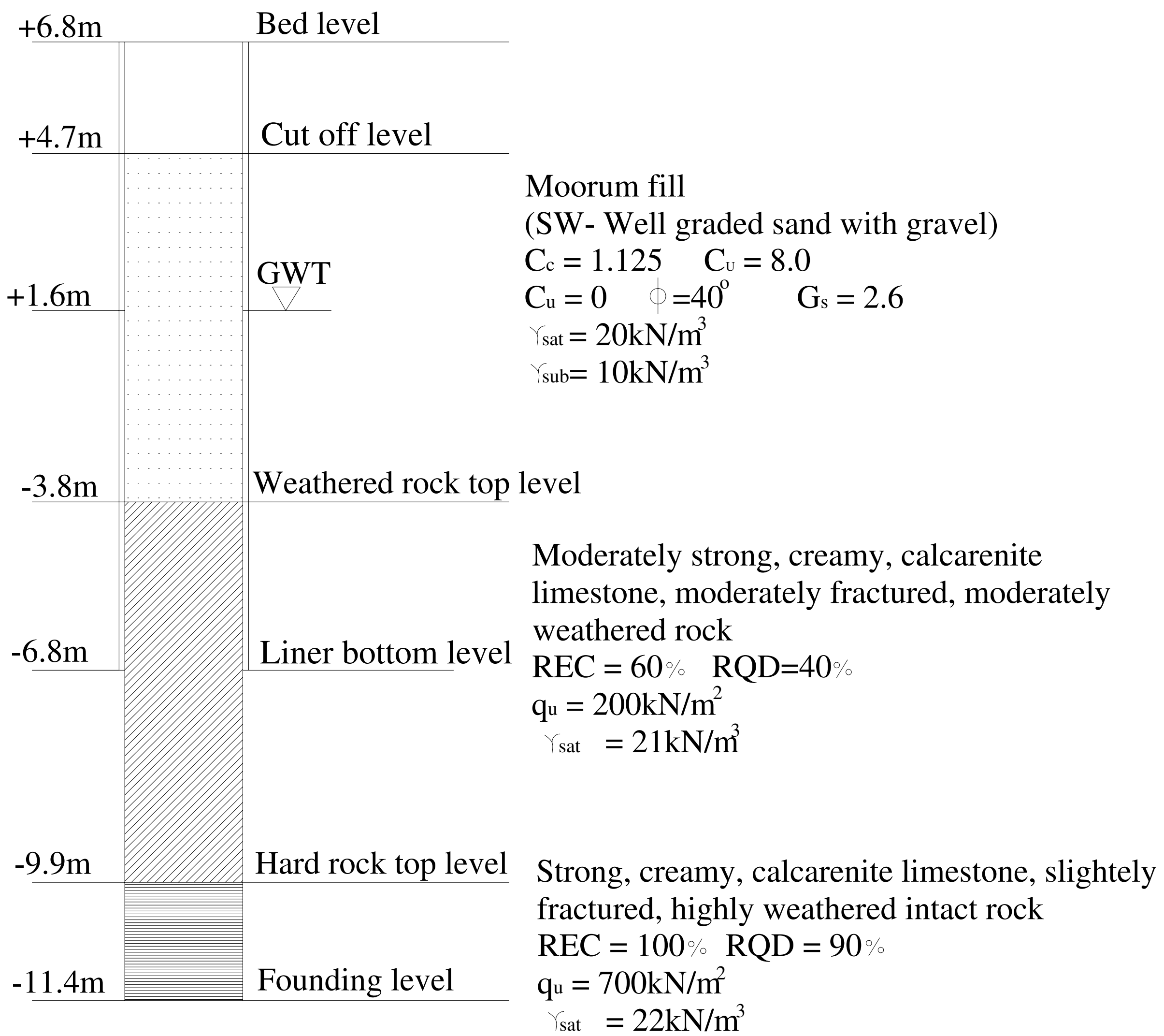


Figure 1. Typical Soil Profile at Pile Load Test Location.

GEOTECHNICAL DATA

The geotechnical investigation at the site was carried out prior to the placement of fill at the site. Standard penetration tests (SPT) were carried out at several locations of the study area in accordance with ASTM D-1586 to understand the stratigraphy. Representative undisturbed and disturbed soil samples were collected for laboratory testing. When rock was encountered, coring was performed to determine the rock type and structure. The cores were obtained as per ASTM D-2113 procedure using swivel-type double tube core barrel. The physical characteristics of selected and representative soil samples were tested adopting the latest versions of ASTM, which are detailed below.

Particle size analysis of soils was done in accordance with ASTM D-422. Soils were classified following ASTM D-2487. Liquid and plastic limits of finer soil samples were obtained as per ASTM D-4318. Specific gravity tests were conducted in accordance with ASTM D-854. Direct shear tests were conducted on representative samples in accordance with ASTM D-3080. Unconfined compressive strength (UCC) tests of fine-grained soils were determined as per ASTM D-2166. Undrained shear strength of soft fine-grained soils were obtained by field vane shear test as per ASTM D-2573. Compressive strength of rock cores was performed as per ASTM D-2938.

GEOTECHNICAL INVESTIGATION AT PILE LOAD TEST LOCATION

A typical soil profile at the location of pile load test is shown in Fig. 1. The top 10.5 m consist of filled moorum soil, followed by rocky formation. The index and engineering properties of the encountered soils and rock are as follows.

Moorum Soil

A typical grain size distribution curve of moorum fill is shown in Fig. 2. The effective particle size (D_{10}) is 0.5 mm, average particle size (D_{50}) is 2.7 mm, Coefficient of Uniformity (C_U) is 8.0, Coefficient of Curvature (C_C) is 1.125 and Specific Gravity (G_S) is 2.6. As per ASTM D-2487, the moorum fill soil is classified as well graded sand with gravel (SW). To obtain the value of angle of internal friction, direct shear tests were conducted on the moorum soil. The average angle of internal friction of the moorum soil was estimated to be 40^o. Typical direct shear test results are presented in Fig. 3.

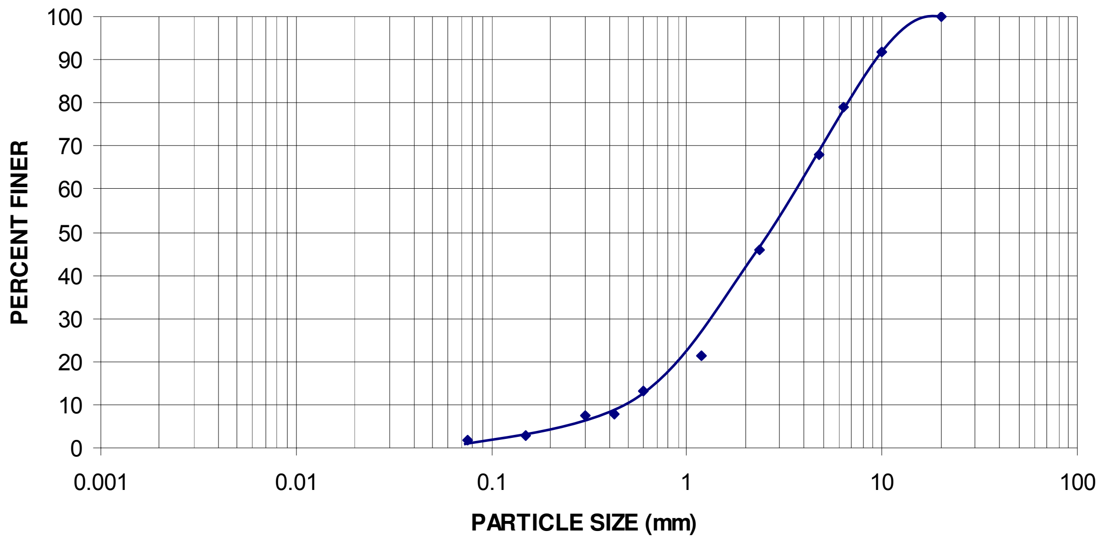


Figure 2. Graind Size Distribution Curve for Moorum Soil.

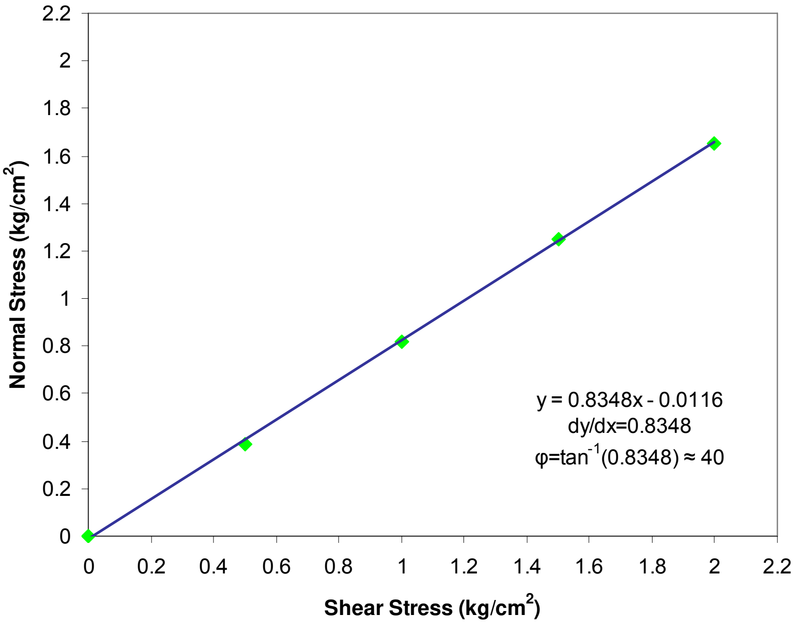


Figure 3. Direct Shear Test Result of Moorum Soil.

Rock Formation

Moderately strong, creamy, calcarenite limestone, moderately fractured, moderately weathered rock with rock core recovery (REC, rock core recovery defined as the ratio of length of core recovered to total length of coring, expressed in terms of percentage) value of 60% and RQD (rock quality designation) value of 40% was encountered at elevations between -3.8 m & -9.9 m. Strong, creamy, calcarenite limestone, slightly fractured, highly weathered intact rock with REC value of 100% and RQD value of 90% was encountered at -9.9 m elevation. For the design load mentioned earlier, the piles were designed with a minimum penetration of 1.5 m into hard rock. Based on the RQD values, the encountered rock is classified as fair to excellent rock.

GEOTECHNICAL INVESTIGATION AT INCLINOMETER TEST LOCATIONS

Fig. 4 shows the typical soil profile at the pile and diaphragm wall panel locations where the inclinometer was installed. The top soil is moorum fill of 9.1 m thickness followed by 4 m soft marine clay with an average SPT (N) value of 3, followed by 2 m thick medium stiff clay with an average SPT (N) value of 8, followed by 7 m thick , very stiff clay with an average SPT (N) value of 18. The clay is underlain by 3 m thick hard marine silty clay with an average SPT (N) value of 32 which in turn is underlain by strong, creamy, calcarenite limestone, highly weathered intact rock with REC value of 100% and RQD value of 100%. Based on the RQD values, the encountered rock formations are classified as excellent rock. The index and engineering properties of the moorum soil are identical to the pile load test location moorum soil, presented previously. The encountered fine-grained soils index and engineering properties are presented below.

Fine Grained Soils

To obtain the index and engineering properties of the fine-grained soils, disturbed and undisturbed soil samples were collected at different depths. Liquid limit and plastic limit tests were conducted on the collected soil samples, to classify the soils based on the Unified Soil Classification System (USCS). The classification of the encountered fine grained soils is presented in Fig. 5. The undrained shear strength of the encountered fine grained soils was evaluated by conducting in-situ field vane shear tests in the soft clay soil and unconfined compression tests in the collected undisturbed soil samples. Two vane shear tests were conducted at elevations -2.0 m and -4.0 m. The average undrained shear strength of the layer (elevations between -2.0 m & -4.0 m) was obtained as 20 kN/m^2. The vane shear computations are presented in the appendix. The unconfined compression tests were conducted on the undisturbed soil samples collected at elevations of -6.0 m, -8.0 m, -11.0 m, -13.0 m, -15.0 m and -17.0 m. The typical unconfined compressive strength variation plot for elevations of -6.0 m, -11.0 m and -15.0 m are shown in Fig. 6a, 6b and 6c respectively. The average undrained shear strength of lean clay at elevations between -6.0 m and -8.0 m was estimated as 50 kN/m^2. The average undrained shear strength value of fat clay at elevations between -8.0 m and -15.0 m was estimated as 112 kN/m^2. The average undrained shear strength value of silty clay at elevations between -15.0 m and -18.0 m was estimated as 200 kN/m^2. The undrained shear strength variation with elevation is shown in Fig. 7. The unconfined compressive strength computations are presented in the appendix.

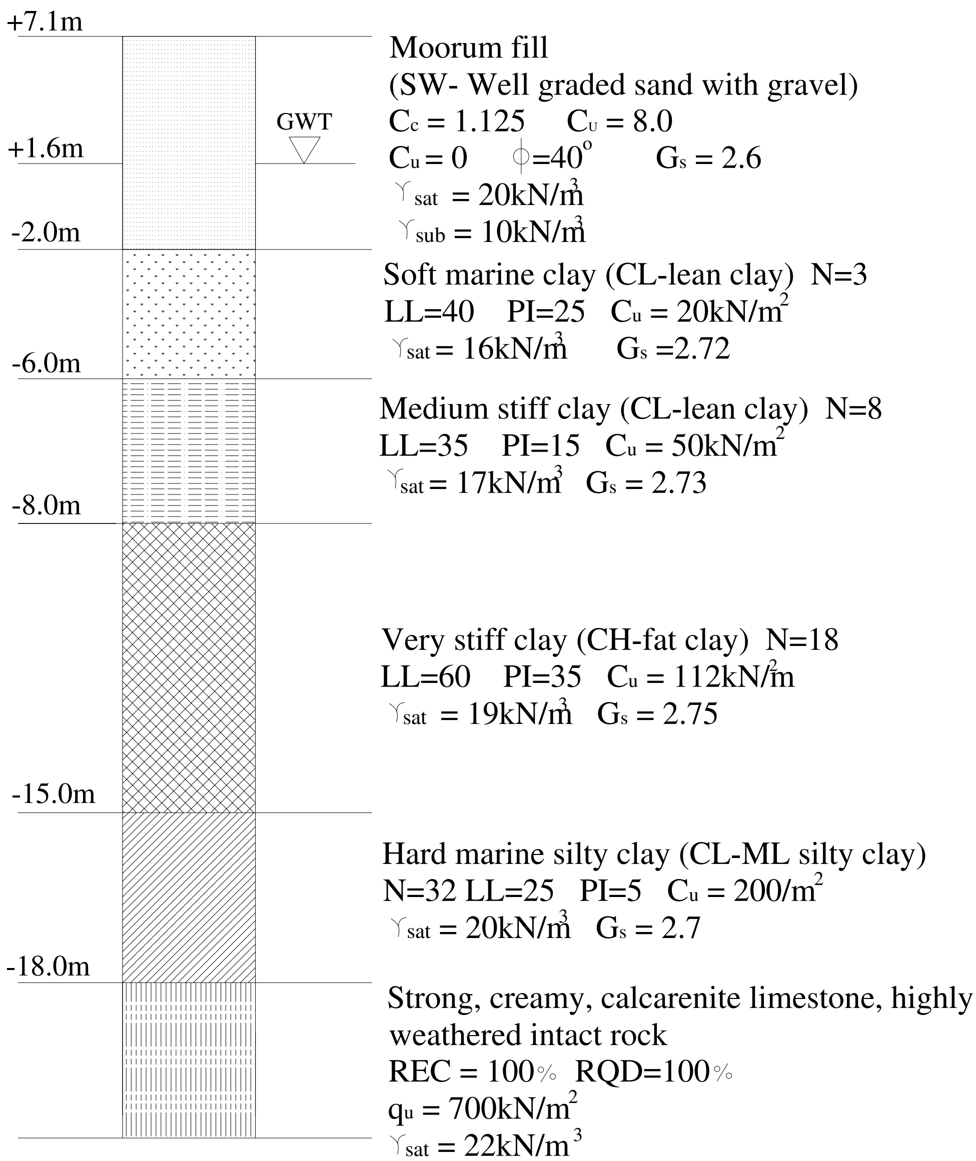
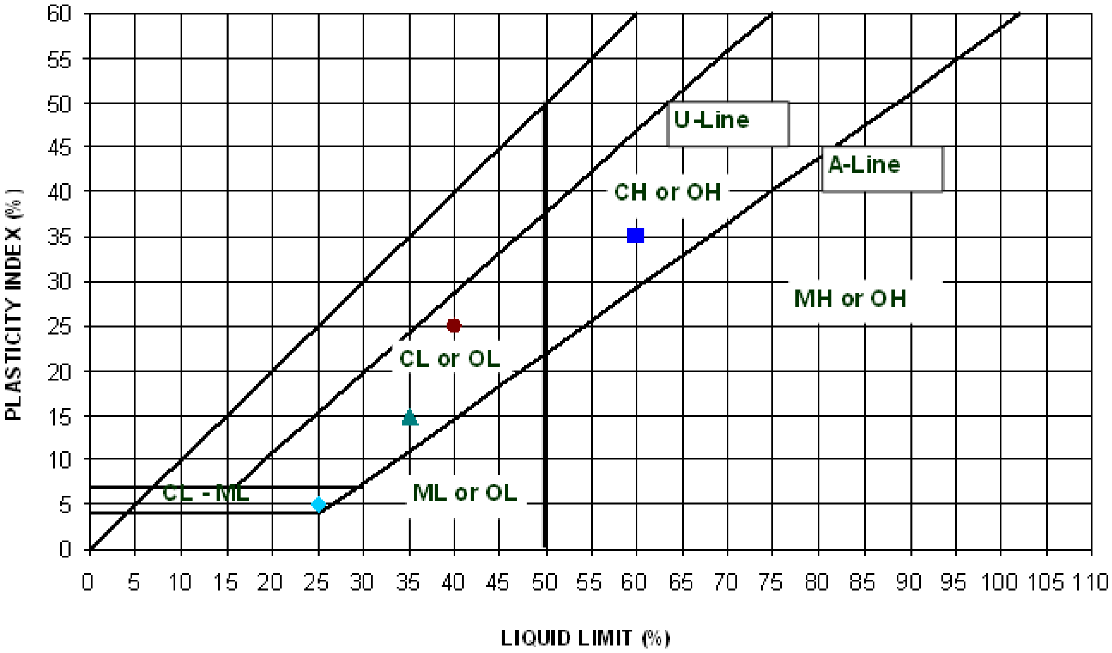


Figure 4. Typical Soil Profile at Inclinometer Test Locations.



Elevation (m)	Symbol	LL (%)	PI (%)	Passing # 200 Sieve, (%)	USC Classification
-2.0 to -6.0	●	40	25	70	CL (Lean Clay)
-6.0 to -8.0	▲	35	15	60	CL (Lean Clay)
-8.0 to -15.0	■	60	35	80	CH (Fat Clay)
-15.0 to -18.0	◆	25	5	40	CL-ML (Silty Clay)

Figure 5. Classification of fine grained soil (ASTM D2487)

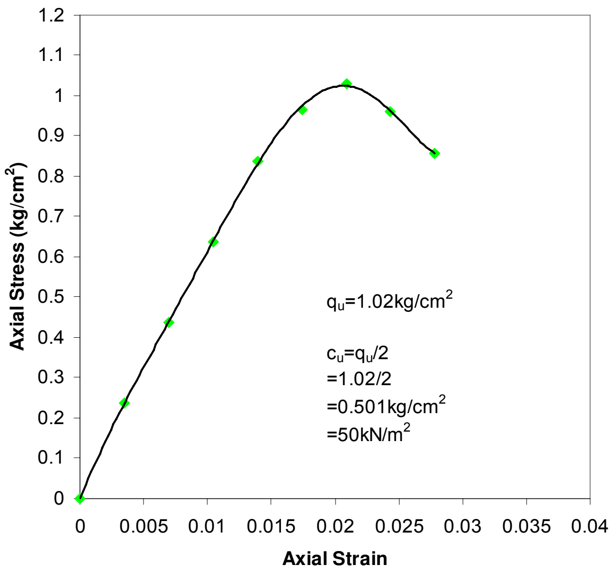


Figure 6a. Unconfined Compression Test Result at -6.0m depth.

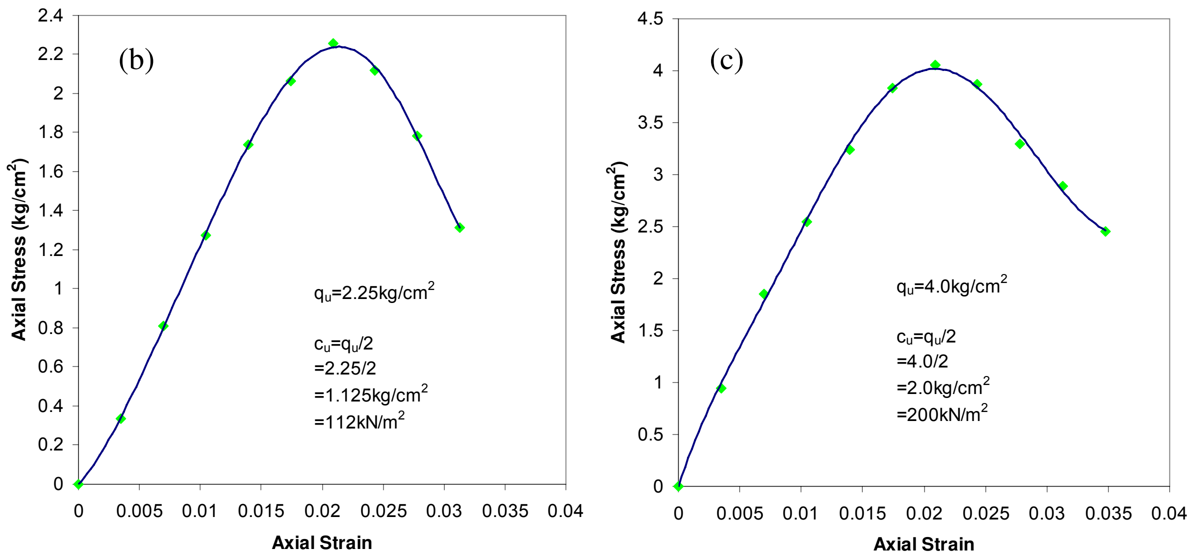


Figure 6b. Unconfined Compression Test Result at -11.0 m depth; 6c. Unconfined Compression Test Result at -15.0 m depth.

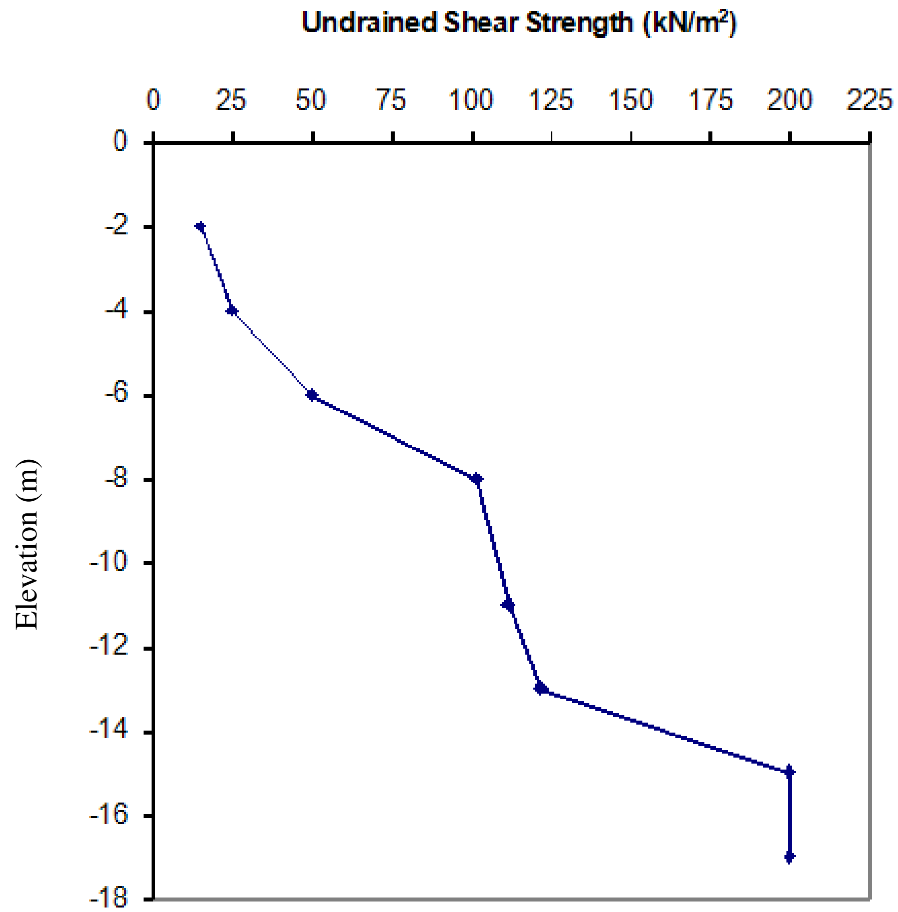


Figure 7. Variation of Undrained Shear Strength with Depth.

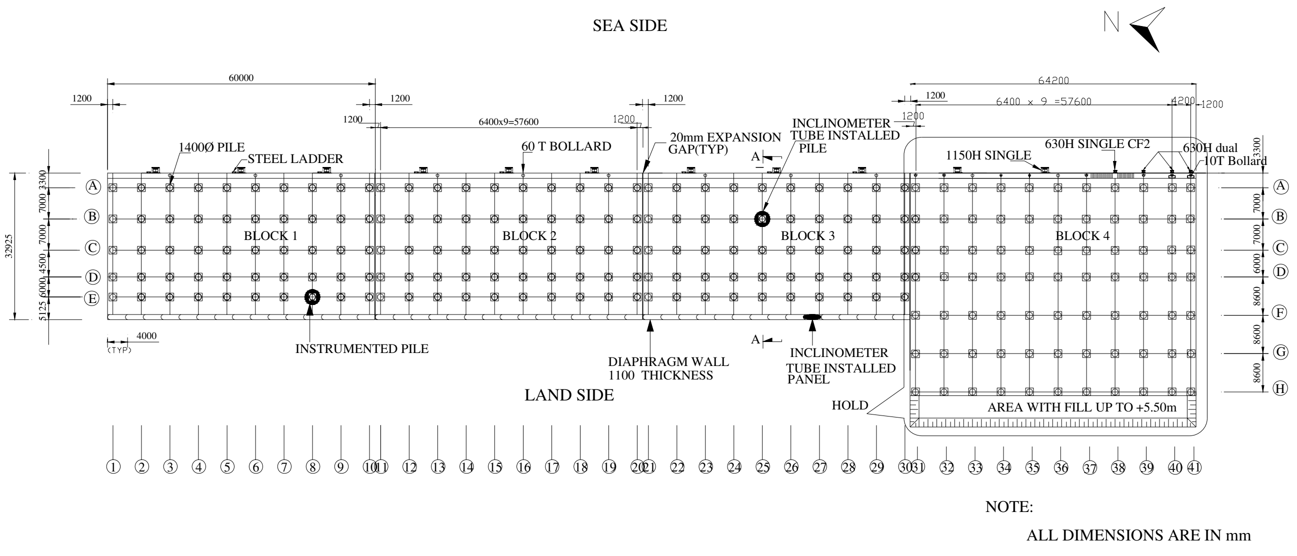
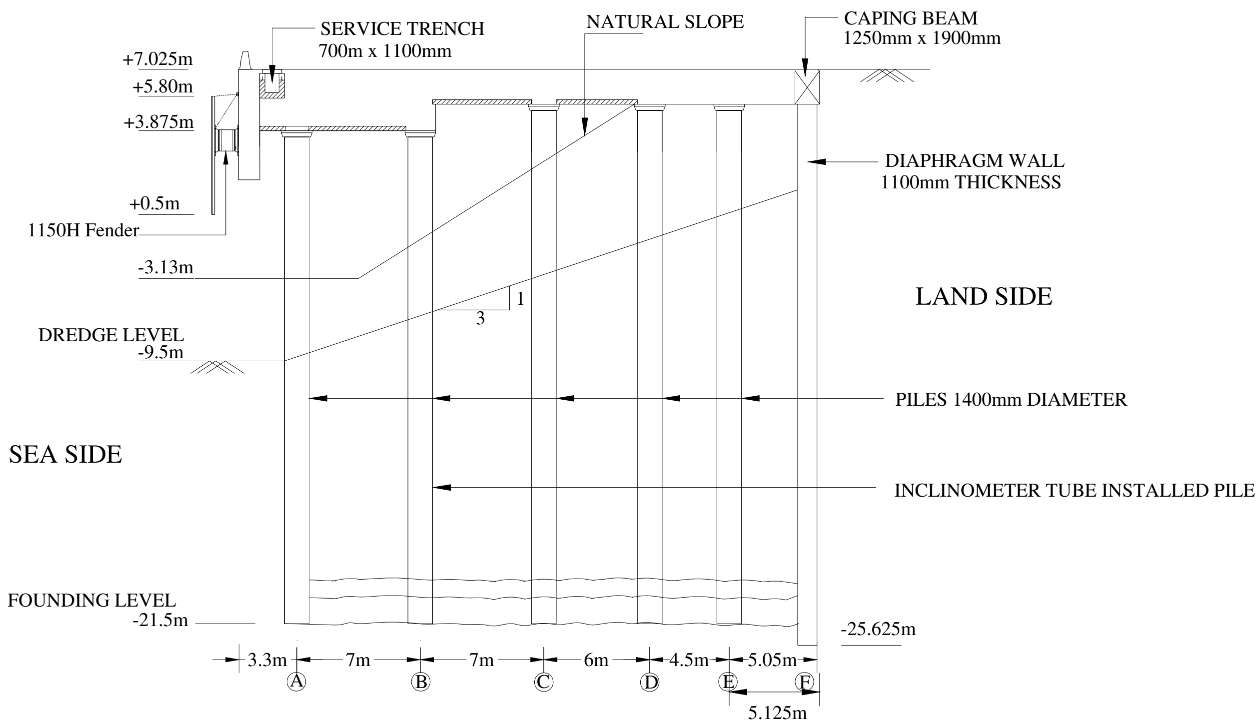


Figure 8. General Layout of Berthing Structure.



Section A-A

Figure 9. Typical Cross Section of Berth.

DETAILS OF THE STRUCTURE

The general layout of the berthing structure and a cross section of the berth is shown in Fig. 8 and Fig. 9 respectively. The total length of the berth is 252 m and the width is 33 m. The berth is supported by both piles and the diaphragm wall. The diaphragm wall thickness is 1100 mm and extends to an elevation of -25.7 m. The construction of guide walls for the diaphragm wall is shown in Fig. 10. The diaphragm wall is constructed with the appropriate number of panels and each panel has a length of 5.0 m. Bored cast in-situ concrete piles of 1400 mm diameter are used and the founding elevation is varied between -11.4 m and -22.0 m based on the soil profile. The axial load test pile is terminated at an elevation of -11.4 m and the instrumented with inclinometer pile is terminated at an elevation of -22.0 m. The construction sequence of piles is shown in Fig. 11. The bored cast in-situ concrete piles were constructed using piling gantry. The natural seabed level at the structural location is +0.0 m elevation. To satisfy the berthing facility of the vessel, the ground level is required to be dredged up to an elevation of -9.5 m. The dredging was done in three stages, in each stage 3.0 m thick material is dredged. The inclinometer readings were collected when the water elevation in front of the structure was –3 m (prior to dredging) and -9.5 m (after -9.5 m dredging). Two sets of readings were collected at – 9.5 m elevation, one immediately after dredge level reached an elevation of 9.5 m and the other 3 months after the first set of readings to evaluate time effects.

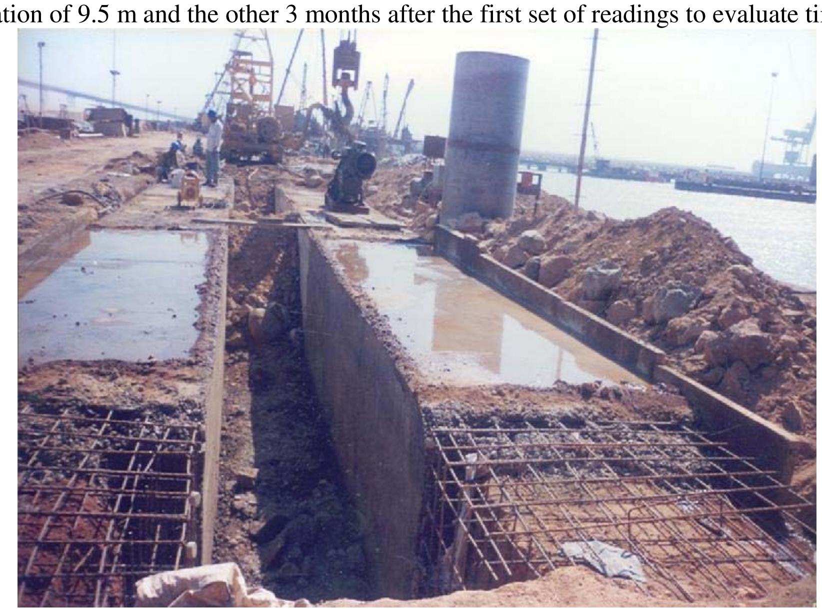


Figure 10. Construction Of Guide Walls For Diaphragm Wall.

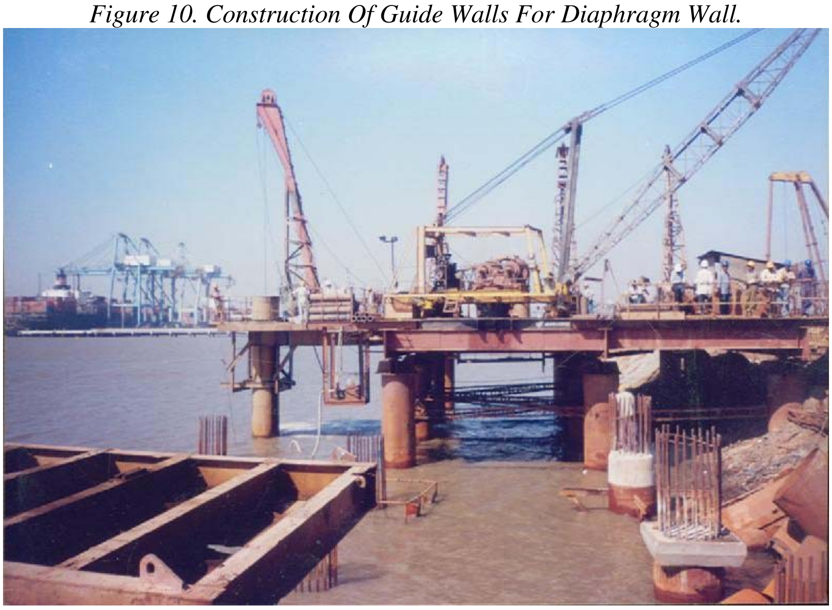


Figure 11. Construction of Piles.

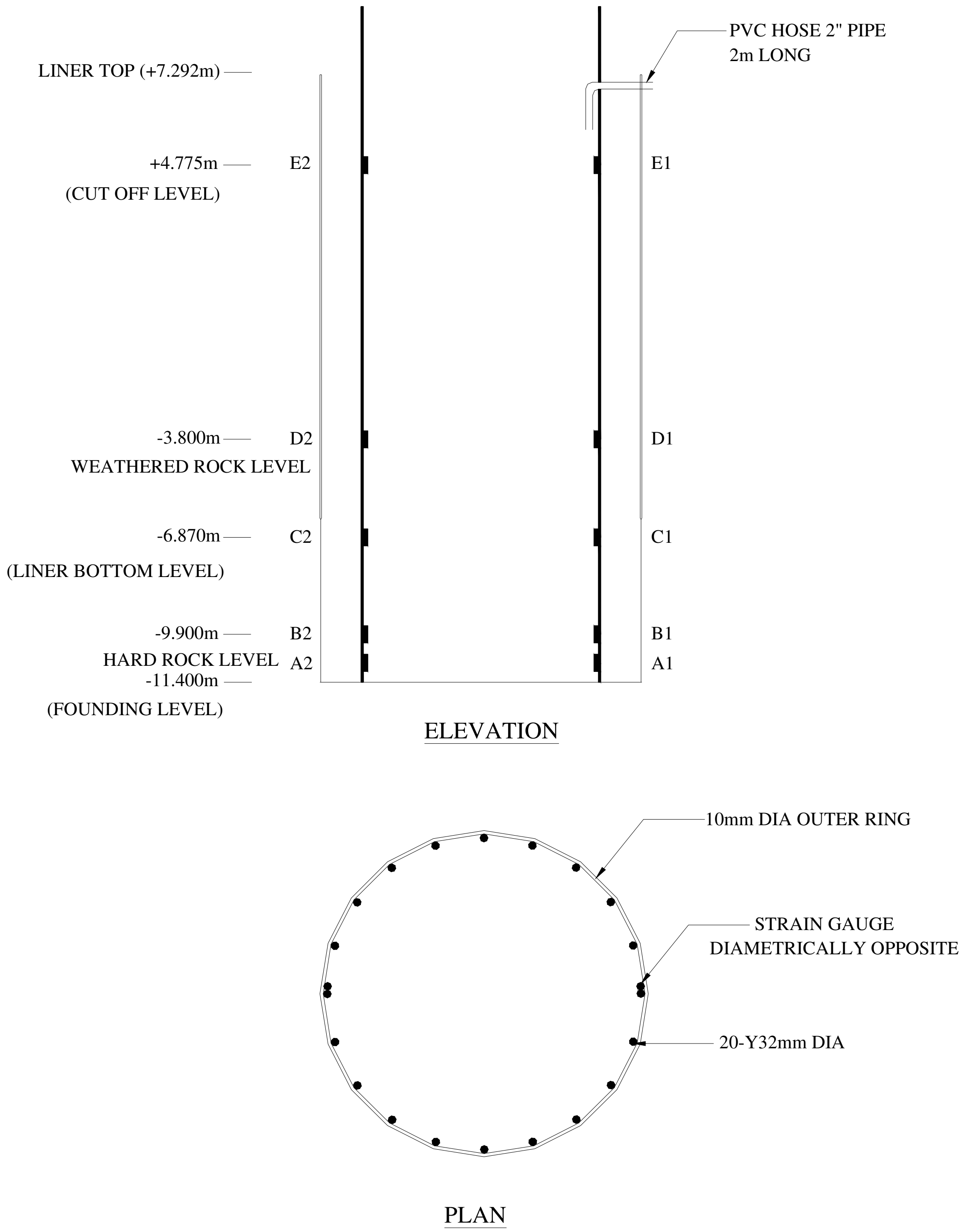


Figure 12. Pile Instrumentation with Strain Gauges.

AXIAL LOAD TEST PILE INSTRUMENTATION

The test pile is instrumented with electrical resistance type strain gauges. The gauges are attached at five different elevations as shown in Fig. 12. Two diametrically opposite main reinforcement rods are selected to mount the strain gauges. After mounting the gauges, the electric cable from each gauge is safely secured along the reinforcement bar to minimize the possibility of damage by the Tremie pipe during concreting. The cables of the gauges on the same reinforcement rod are bundled together and tied with a plastic wire to the reinforcement rod every 30 cm. The electric cables have a minimum length of 5 m beyond the cut off length to enable connection to the measuring instrument during the load test. All the strain gauges are aligned vertically.

AXIAL LOAD TEST ARRANGEMENT

For the axial load test, the load applied on the test pile is obtained by the reaction forces from 4 pre-stressed rock anchors. Each anchor has a capacity of 2812.5 kN. The load on the pile is applied via 4 hydraulic jacks each of 2500 kN capacity. The settlement of the pile is recorded by 4 fixed dial gauges located at the same distance from the center of the pile. These dial gauges are installed on two datum bars with immovable support, at a distance of 3.6 m from the edge of the pile. A schematic view of the pile load test arrangement is shown in Fig. 13.

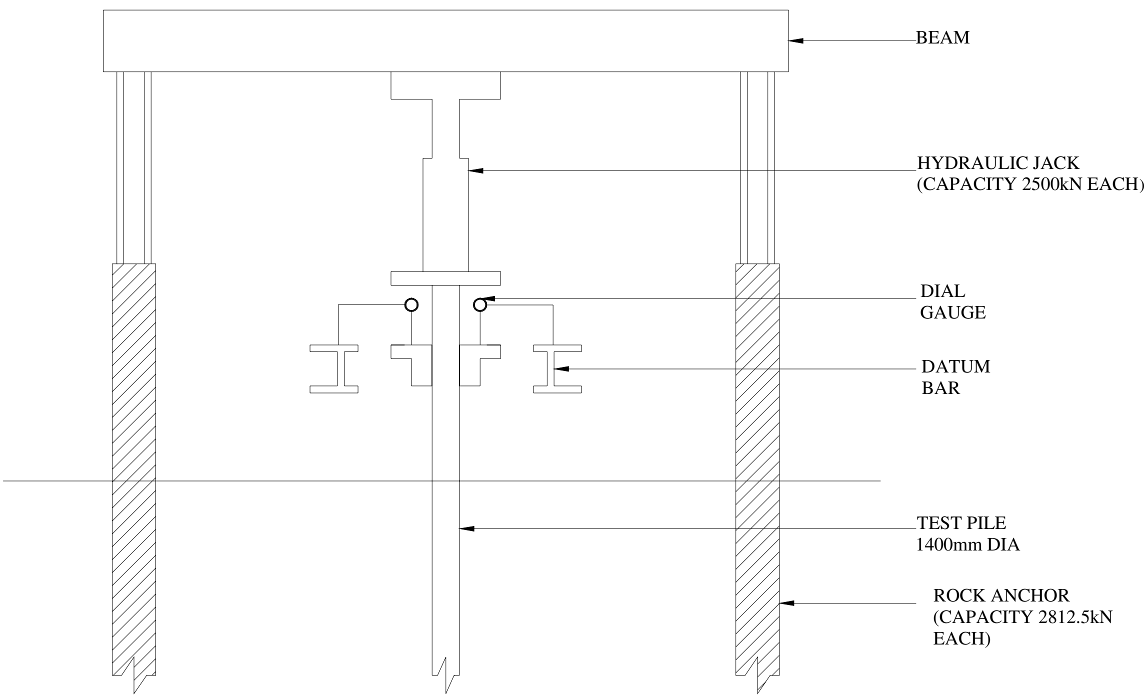


Figure 13. Schematic View of Pile Load Test Arrangement.

AXIAL LOAD TEST PROCEDURE

Four hydraulic jacks were placed concentrically on the test pile. The test was conducted according to the recommendations in IS 2911 (Part- 4) – 1985. The test load of 9000 kN is reached in a total of eight stages. Dial gauge readings were taken at regular intervals of 10 minutes until the readings remained constant for 10 minutes. After maintaining the test load for 24 hours, unloading was performed in 5 stages and readings were recorded as specified in IS2911 (Part-4) 1985. During the load test, the axial strain of the pile at each level was measured by the read out unit for every increment of loading. Strain gauge readings were taken 10 to 15 minutes after the load increment was applied. To estimate the mobilization of axial load along the pile shaft, the measured strain readings were converted to equivalent axial load by using a calibration constant. To obtain the calibration constant, one strain gauge was embedded in a concrete cylinder of 150 mm diameter and 300 mm long. The strain gauge was oriented such that the axis of strain gauge was parallel to the concrete cylinder axis. The cylinder was cured for 28 days and then tested in compression. The strain readings were observed to be almost linear with applied load. Fig. 14 shows the load as a function of strain. From the graph, the calibration constant of 55.75 kN was obtained, 1 micro strain being equal to 55.75 kN load.

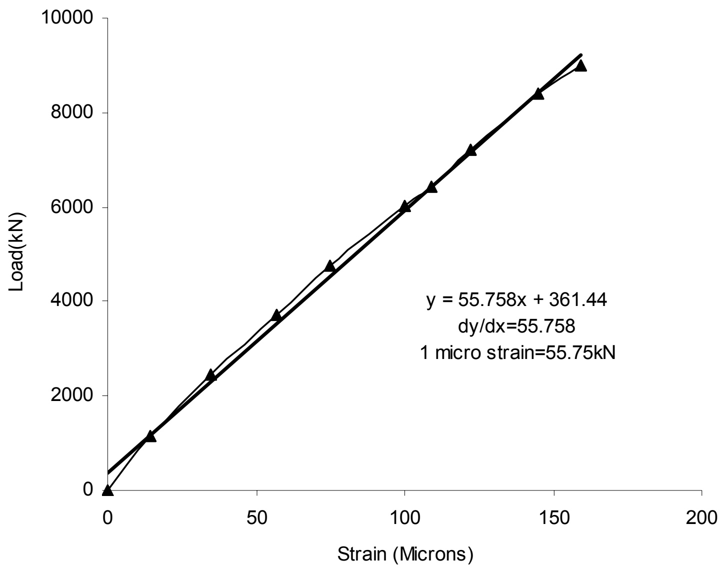


Figure 14. Strain Gauge Calibrations.

MONITORING OF SUBSURFACE MOVEMENTS BY INCLINOMETER

To investigate the lateral displacement of the berth due to lateral soil movements caused by dredging, inclinometers were installed in one diaphragm wall panel and in one pile. A 22 N weight and 700 mm length with 500 mm measuring length inclinometer is used for the present study. Data collected are analyzed and processed to measure the lateral displacement, rate of movement, and the magnitude and direction of movement.

INSTALLATION OF INCLINOMETER TUBE

The inclinometer tube is made of PVC, which is very flexible and can easily match the deformation profile of the diaphragm wall and pile. During casting of the diaphragm wall panel and pile, an inclinometer tube is placed with the reinforcement gauge. The location of the inclinometer tube is selected such that it is away from the tremie pipe location. The length of the inclinometer tube above the cut off level was sealed and protected by a 150 mm diameter rubber hose. The annular gap between the rubber hose and the inclinometer tube is filled with bentonite mud to ensure that no concrete enters the hose pipe during concreting.

TEST RESULTS AND DISCUSSION

Axial Pile Load Test

The pile was tested up to 1.5 times the design load, i.e., up to 9000 kN. The load settlement curve is shown in Fig. 15. As per IS 2911 (Part-4) 1985, the maximum settlement at working load (1.5 times design load) may not exceed 12 mm. In the present study, the total settlement was 3.9 mm, rebound was 3.6 mm and net settlement was 0.3 mm, indicating that the pile behaved well and has not reached its capacity. The pile load test results were compared to the Gandhi et al. (1987) pile load test results, which are also shown in Fig. 15. As presented in that paper, the pile load test was conducted on a 6.5 m long and 0.5 m diameter pile. Out of the 6.5 m, 0.5 m are socketed in hard rock strata. The test was conducted at the NALCO project site, Angul in India. The soil profile at this location includes 3.0 m of red soil with an average SPT (N) value 35, followed by yellowish grey weathered rock with SPT (N) values of about 100. Dark grey hard rock is present at 6.0 m below the ground level. The subsurface stratigraphy of the present study area (JNPT, Mumbai) and Gandhi et al. (1987) study area (NALCO project site, Angul) are not the same and thus, the load-settlement response of the test piles is different.

Fig. 16 shows the load resisted by each soil layer. The maximum load is mobilized from an elevation of -6.87 m to -9.9 m, where the weathered rock is present. The mobilization of frictional resistance is not significant at the top layer of moorum fill. Nearly 4000 kN load is mobilized in the weathered rock, 2000 kN mobilized in hard rock, and 2000 kN mobilized in the moorum fill strata under a total applied load of 9000 kN. Only 1000 kN load is resisted by end bearing by the hard rock.

Fig. 17 shows the load shared by each layer for all applied load increments. For all cases except the load increment of 8400 kN and 9000 kN, the frictional resistance was significantly increased in all soil layers compared to the end bearing resistance. For the test load of 9000 kN, the end bearing resistance was higher than the frictional resistances of the moorum fill and the hard rock. However, the mobilization of end bearing was less than the frictional resistance in weathered rock.Nearly 75% of the load is resisted by friction and 25% of the load is resisted by end bearing. The weathered rock contributed about 20% to 45% of the total pile resistance.

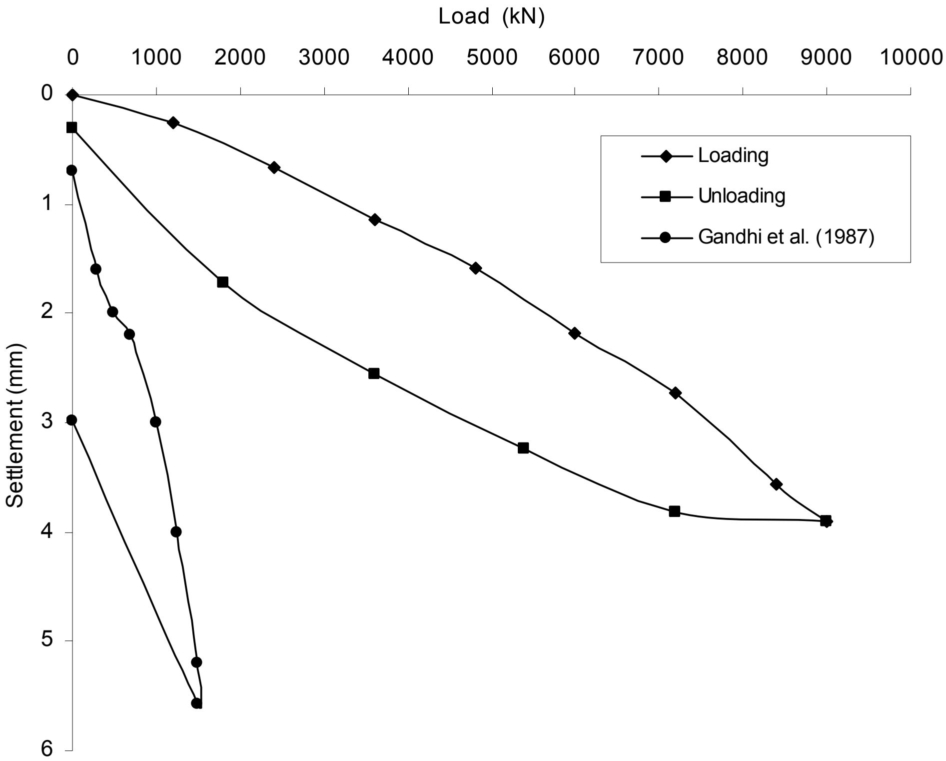


Figure 15. Load Settlement Curve.

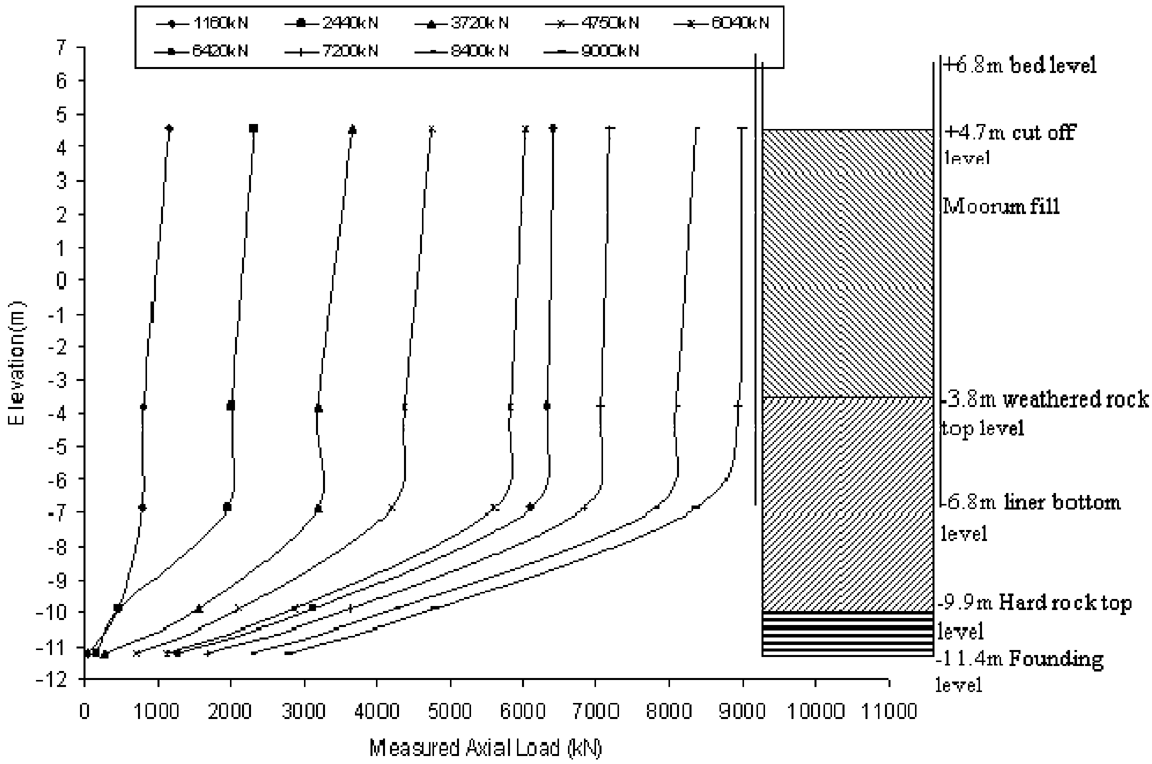


Figure 16. Mobilization of Axial Load.

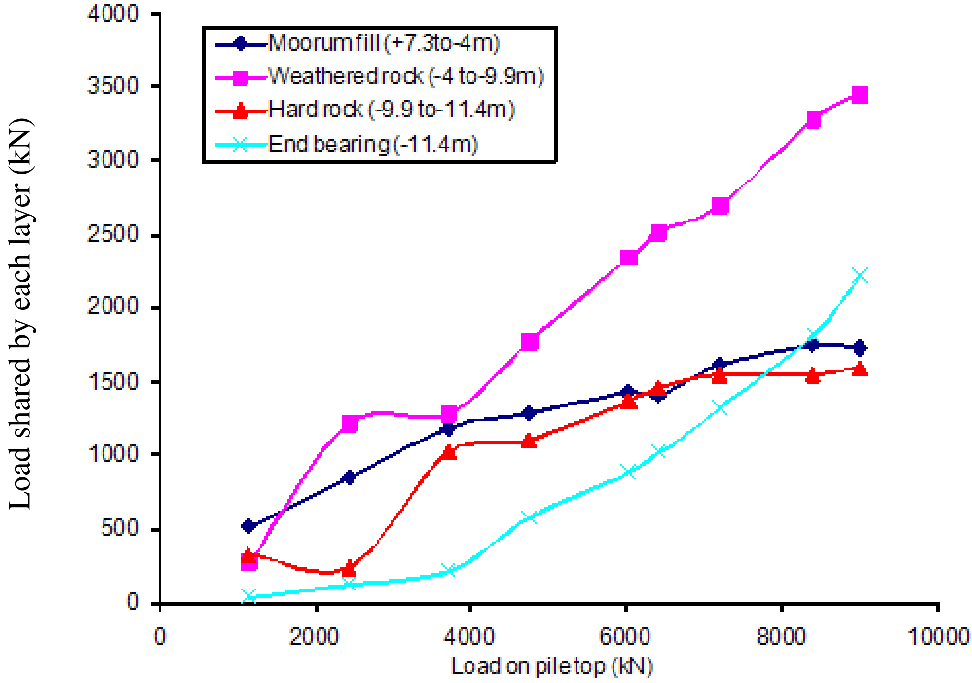


Figure 17. Axial Load Shared by Each Layer.

Inclinometer Measurement

After construction of the full berth, the inclinometer readings were collected in 3 stages. First set of readings were taken after construction of the berth and prior to dredging. This reading represents the initial position of the diaphragm wall and pile. The second readings were collected immediately after dredging to an elevation of -9.5 m. The difference between the second and first reading is the deflected shape of the diaphragm wall panel and pile. The third readings were collected 3 months after dredging to an elevation of –9.5 m. The difference between the third and second reading is shown in Fig. 18 and 19.

It can be observed that for the diaphragm wall immediately after -9.5 m dredging, the maximum deflection is 15.2 mm at the level of +4 m, while at the same level the maximum deflection increased to 17.3 mm after three months. The increase in deflection is probably due to the consolidation and associated pore pressure dissipation effects of the soft clayey soil. The same behavior was observed for the pile also. For both the pile and the diaphragm wall the deflection from –16 m elevation and lower is small compared to the upper layers due to the lower hard layers of marine silty clay and rock.

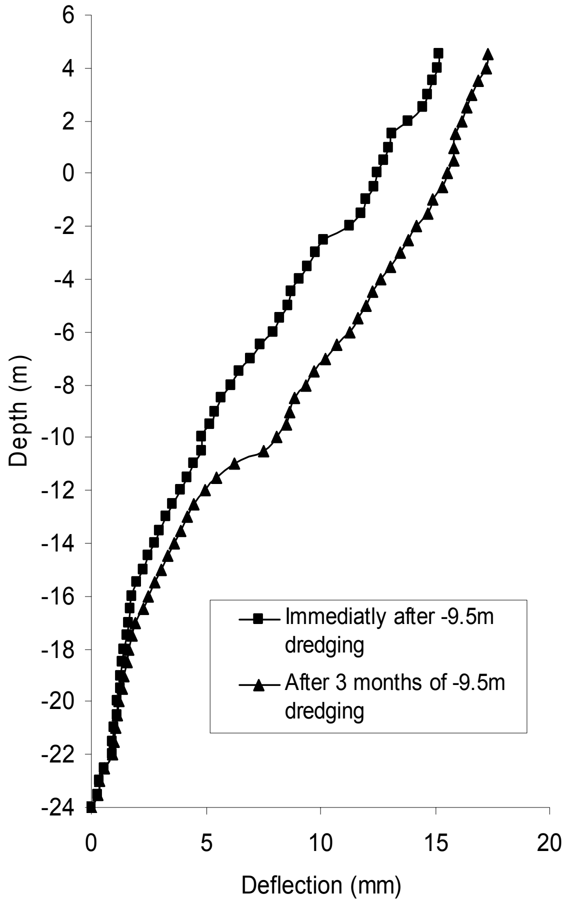


Figure 18. Deflected Shape of Diaphragm Wall.

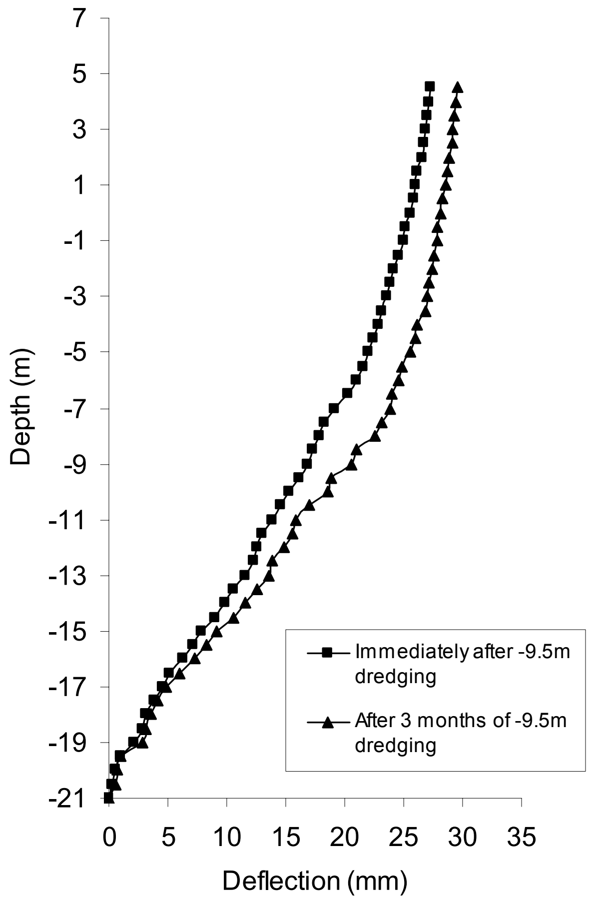


Figure 19. Deflected Shape of Pile.

CONCLUSION

Axial load test

(1) The total settlement of 3.9 mm was within the settlement limit of 12 mm as per IS 2911 (Part-4) 1985, which indicates that the pile has not reached its capacity.

(2) The weathered rock at the site offered substantial frictional resistance, i.e. about 20% to 45% of the total resistance.
(3) A maximum of 75% load is mobilized by frictional resistance and the remaining 25% load is mobilized by the end bearing resistance. At higher loads the contribution of the end bearing resistance increases.
(4) The use of strain gauges was successful in measuring the axial load distribution in concrete piles.

Inclinometer Measurement

(1) The maximum deflection of 30 mm was measured in the berthing structure after dredging to an elevation of -9.5 m. The permissible lateral movement of the berthing structure was 150 mm and thus the deflections observed were considered small.
(2) About 87% of the maximum deflection was mobilized at the soft top layers, probably due to the presence of the underlying harder ground materials.

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