Geotechnical Characterization of the Eardo Seabed for Offshore Pile Foundation Design

해양말뚝 기초설계를 위한 이어도 해저지반의 특성화

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요 지

한국해양연구소(KORDI)는 후구로(Fugro Int.)사의 도움으로 마라도에서 약 152km 떨어진 지점에 건설 예정인 이어도 해양관측기지 건설을 위해서 해저지반의 특성조사를 수행하였다. 지반조사의 근본 목적은 해양과학기지가 설치될 이어도의 해저지반의 공학적 특성을 파악하고 조사자료를 이용하여 고정식 해양구조물의 기초설계를 하기 위한 것이었다. 본 논문에서는 해저지반조사의 상세한 설명과 고정식 해양구조물의 기초설계에 필요한 설계상수를 산정하는 방법에 대해서 토론하고자 한다. 연구결과로는 해저 현장토질의 특성을 고려한 해양말뚝의 기초설계에 필요한 지반설계 상수를 제안하였다

Abstract

Korea Ocean Research & Development Institute(KORDI) conducted an offshore geotechnical investigation for the Eardo Ocean Research Station with the help of the Fugro International Limited at a site location approximately 152 km away from Mara Island, Korea. The primary purpose of the geotechnical investigation was to obtain information on soil and foundation conditions, and to develop foundation design data for a fixed offshore observation platform. This paper discussed the details of the geotechnical investigation and the foundation design recommendations for the Ocean Research Station. Clear recommendations were proposed for the foundation type of driven pile considering the existing soil conditions.

Keywords : site characterization, pile design, Eardo, foundation, soil properties.

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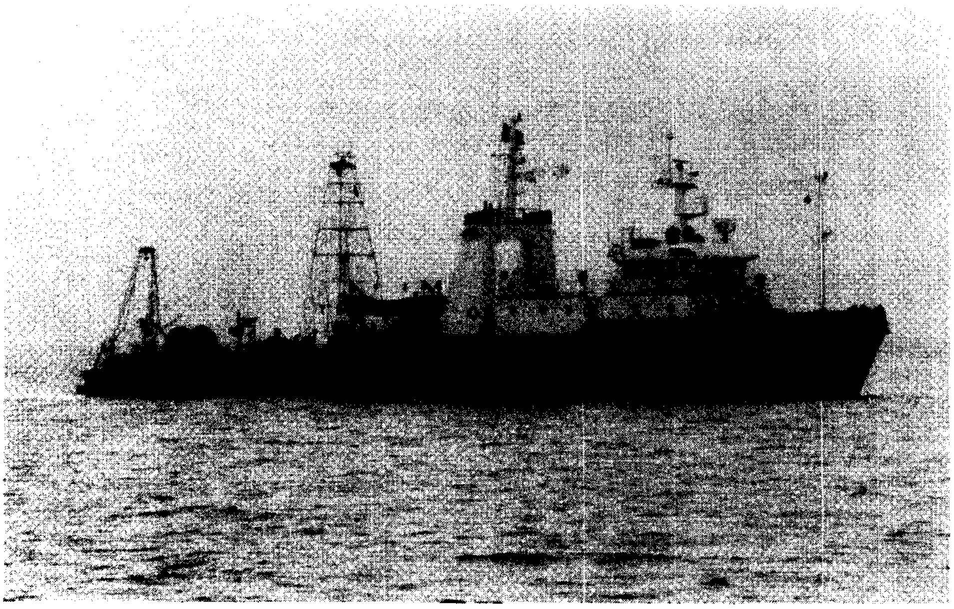


Fig. 1 Drilling Vessel M/V Kan 407

1. Introduction

The geotechnical investigation work was carried out at Eardo, the southwest of Cheju, approximately 152km away from Mara Island, Korea.

The vessel used for the field investigation is an offshore geotechnical investigation vessel , M/V Kan 407, capable of self propelling and anchoring as shown in Fig.1. It is equipped with navigation, positioning and drilling equipments. The drilling was carried out using a rotary rig positioned over the center-well of the drill vessel. The drill hole with a minimum diameter of 114 mm was formed by water flushing and casing down to 20m penetration. Casing sizes ranging from 17.78mm(7inch) to 11.43mm(4.5inch) were used to provide support for the drill hole. Below 20 m penetration, the drill hole was formed by open hole bentonite flushing down to terminating depth of 51.4m.

Soil samples were recovered using various types of sampling equipments including vibrocorer, U76 percussion sampler, double tube Mazier sampler and single tube core barrel (drilling no flushing). Sampling intervals were generally at 1.5m down to 7.5m penetration; at 3.0m down to 23.5m penetration ; continuously down to a termination depth of 51.4m. Upon recovery, the soil sample was examined, logged and tested onboard. Selected soil samples were packed in waxed tube and/or air-tight plastic bag for further testing in the onshore laboratory.

The vibrocorer was used to recover soil sample near the seabed level. Percussion sampling consisted of an open tube steel sampler attached to drill rod was driven into the soil by a 69kg hammer with a drop of approximately 1.5m to achieve a maximum penetration of 0.45m. The

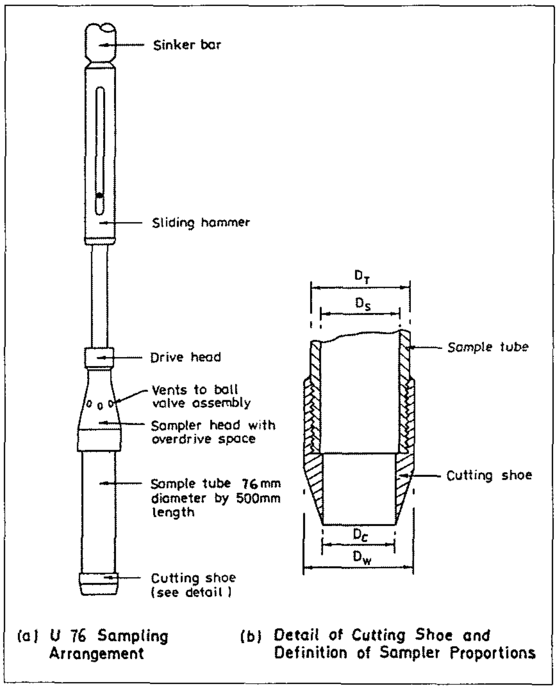


Fig. 2 Descriptions of Sample Tube Used

sample tube shown in Fig.2 is 76mm OD and 500mm in length, which is connected to a sampler head with vents to ball valve assembly, which allows free exit of water above the sample. A Mazier sampler is a retractable double tube core barrel with a 74mm diameter PVC tubing liner. The outer tube of the sampler is fitted with a drill bit and advanced into soil by rotary drilling. The inner tube of the sampler is fitted with an inner barrel cutting shoes protruding below the drill bit.

During drilling the inner tube is kept stationary by the retractor spring and the inner swivel assembly of the sampler. Flush fluid is provided in the gap between the inner and outer tube. During sampling the soil sample is pushed into the PVC tube liner by the advancing action of the outer tube. A single tube core barrel is basically a 3 inch diameter open steel tube fitted with a vent to ball valve assembly at the top and to a drill bit at the bottom end. During sampling the core barrel is advanced into the soil by drilling. No flushing fluid is provided. The soil conditions at the proposed platform site were investigated by drilling and sampling at one boring location only to a depth of 51.4m below the seafloor.

2. General Soil Descriptions

The laboratory soil testing program for this study was designed to evaluate the pertinent index properties of the foundation soils and determine the appropriate engineering soil properties for pile design analyses. During the field operation, the soil samples were examined and visually classified.

Natural water content determinations and unit weights measurements were performed on selected soil samples. Shear strength estimates were performed on cohesive soil samples using a torvane device. After onboard testing and visual examination, representative portions of the soil samples were sealed in airtight containers including waxed tubes and sealed plastic bags for further testing in the laboratory onshore. The onshore laboratory test program was designed to supplement and verify field information and to determine the static engineering soil properties of the foundation soils.

Standard laboratory tests, including wet density determinations, water content determinations, atterberg limit tests, particle size analyses, UU triaxial compression tests and direct shear box tests were also performed. Plastic and liquid limits, collectively termed the atterberg limits, were determined for selected cohesive soil samples to provide classification information. Natural water content and wet density of soil samples were measured in the field as well as in the laboratory by weighing soil samples of known volumes before and after oven-drying. Sieve analyses and hydrometer testing were performed on selected granular soil samples to determine the particle size distribution. In this type of strength test, either an undisturbed or remolded test specimen is enclosed in a thin rubber membrane and subjected to a confining pressure at least equal to the computed effective overburden pressure. The specimen is not allowed to consolidate under the influence of this confining pressure prior to testing. The test specimen is then loaded axially to failure at a constant rate of strain without permitting any drainage from the specimen.

The undrained shear strength of the cohesive soil is computed as one-half the maximum observed deviator stress. The angle of internal friction of remolded granular soils such as sand was determined by shear box tests. A multi-stage approach was adopted in the testing. The test specimens were consolidated to pressures of 0.5, 1.0 or 1.5 times the estimated past vertical effective stress. Following consolidation, the specimens were sheared at slow rates to ensure drained conditions. Once the peak shear stress was attained, the specimens were reconsolidated to the next level of stress and sheared.

	Penetration (m)		
Stratum	From	To	Description
I	0.0	1.5	Loose fine to medium sand
II	1.5	8.0	Dense silty fine sand
III	8.0	12.0	Soft to firm low plasticity clay
IV	12.0	23.5	Dense fine to medium sand
V	23.5	31.2	Firm to stiff low plasticity clay
VI	31.2	39.0	Dense silty fine sand
VII	39.0	48.0	Dense silt
VIII	48.0	51.4	Dense fine sand

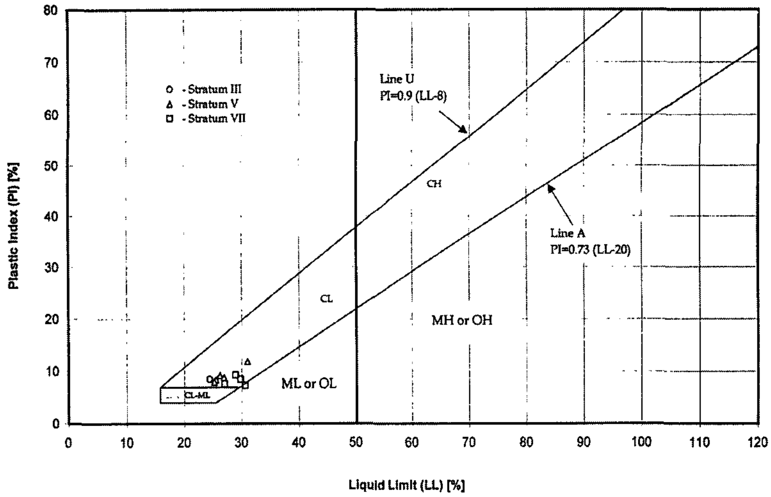


Fig. 3 Plasticity Chart

On the basis of the information from the fieldwork and the results of laboratory tests, the general soil conditions at the soil boring location were interpreted and presented as follows:

The soil stratigraphy is based on the field observations as well as the information obtained from the laboratory tests. Recommendations for pile foundation design contained in this study were developed based on the assumption that soil conditions as revealed by the soil boring are continuous throughout the general area of the platform site. A plasticity chart is plotted in Fig.3 using the plastic limits (PL) and liquid limits data measured. This plasticity chart was constructed on the basis of the Unified Soil Classification System (ASTM D2487-93). On the plasticity chart, the liquid limit is plotted as a function of the plasticity index (PI). As illustrated in Fig.3, the cohesive soil samples from strata III and V are, for most part, lean clay with low plasticity (CL) with the atterberg limits data falling at the lower end of the A-line, near the boundary to CL-ML(silty clay region). For Stratum VII the atterberg data indicate that the soil samples are mostly silt.

In order to develop a profile of in situ vertical effective stress, an estimate of submerged unit weight profile is required. During both the offshore and onshore phases of the investigation, submerged unit weights were measured on cohesive soil samples and where possible, on granular soil samples. This interpreted profile of Fig.4 was used to develop the effective vertical stress profile and also for use in the subsequent engineering analyses. Fig.5 shows the results of undrained shear strengths of the clayey soils encountered in the soil boring evaluated from torvane, unconsolidated-undrained triaxial compression tests as well as the soil classification tests. The granular soil parameters were selected based on their gradation as revealed by grain-size analyses, field observations, and the results of direct shear box tests. Soil deformation characteristics in cohesive soils were studied in this investigation by evaluating the strain corresponding to 50 percent of the maximum deviator stress in an UU triaxial compression test. The strain level is

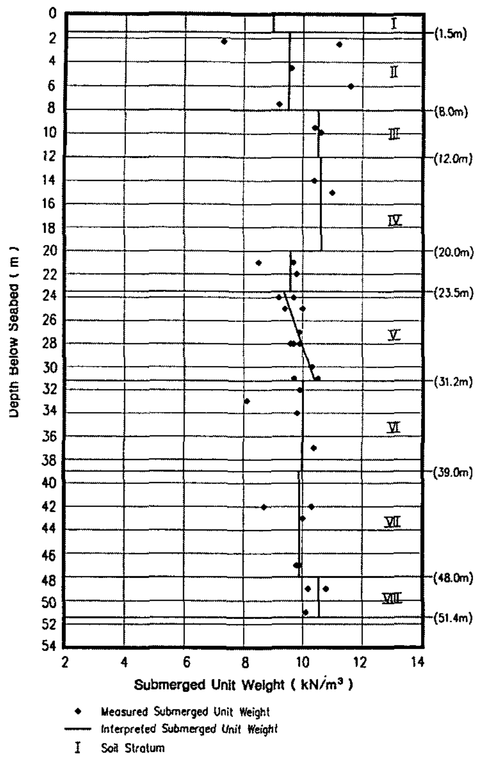


Fig. 4 Submerged Unit Weight of Soils with Depth

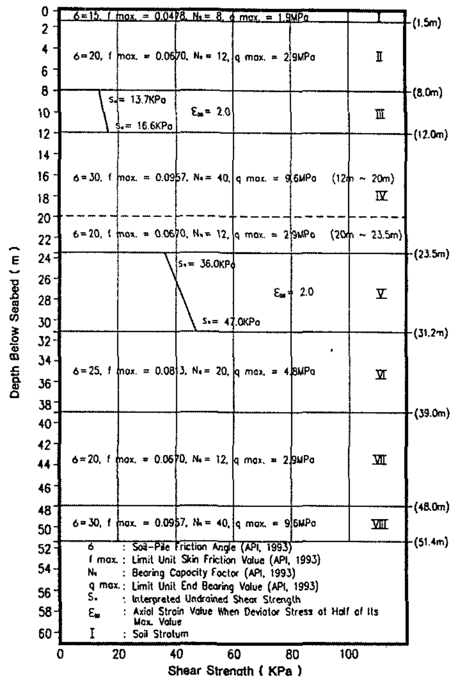


Fig. 5. Undrained Shear Strength of Soils with Depth

denoted as ε_{50} and is one of the parameters that are required to develop the lateral soil pile deflection data (p-y data). Based on the laboratory results and the recommendations of Matlock (1970), the ε_{50} value of 2 percent was selected for the clays in both Strata III and V to sdevelop the p-y data.

3. Pile Design Analyses

Pile design information developed for this study includes ultimate axial capacities, axial load-pile movement, (t-z and Q-z) data, soil resistance-pile deflection (p-y) characteristics. Computation of the ultimate axial capacity of open-ended driven pipe pile was accomplished using the static method of analysis, which consists of methods and recommendations presented in API RP 2A-WSD (1993). In this static method of analysis, the ultimate compressive capacity of a pile for a given penetration was taken as the sum of the skin friction on the pile wall and the end bearing on the pile tip. Also, the analysis assumed that the end bearing was limited to the frictional resistance of a soil plug developed inside the pile.

The unit skin friction on the inside of the pile was assumed equal to that on the outside of the

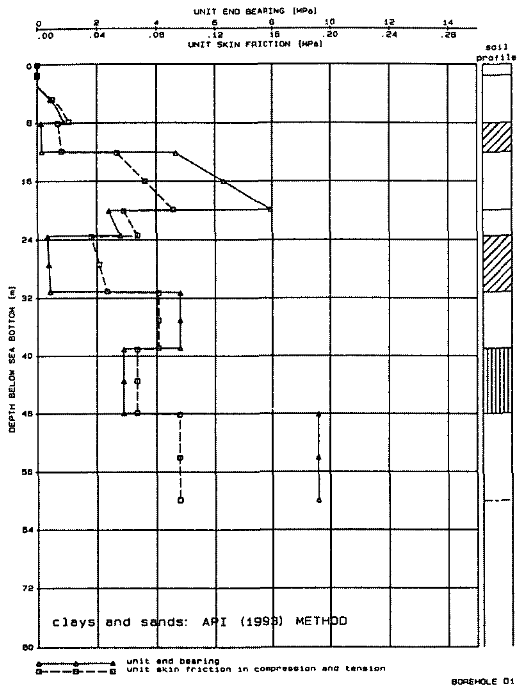


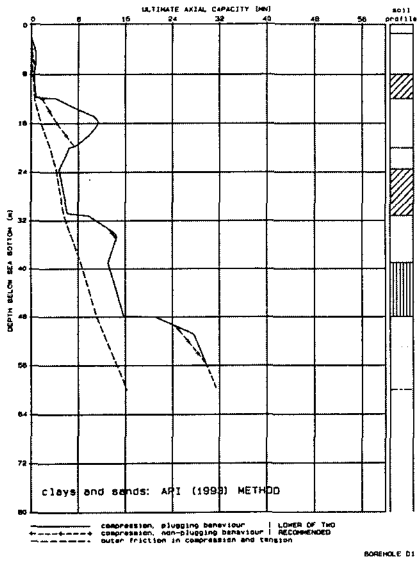
Fig. 6 Unit Skin Friction and Unit End Bearing Values with Depth

pile. The end bearing component was neglected when computing ultimate tensile capacity. The effective weight of the pile and soil plug was not considered in the analysis. The unit skin friction and unit end bearing values plotted in Fig.6 respectively were used to compute the ultimate compressive and tensile capacities of 56inch(1422.4mm), 60inch(1524mm), 72inch(1828.8mm) and 76inch(1930.4mm) diameter open-ended driven pipe piles.

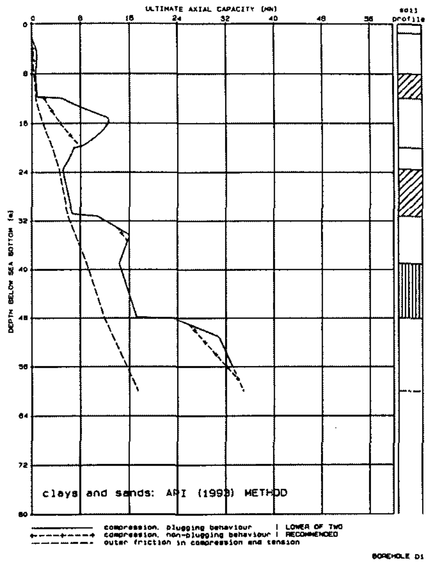
The ultimate capacity curves for the above pile sizes were computed to 60m penetration. This is based on the assumption that the soil condition of Stratum VIII continues beyond the terminating depth of 51.4m to a depth in excess of 80m.

3.1 Pile Penetration

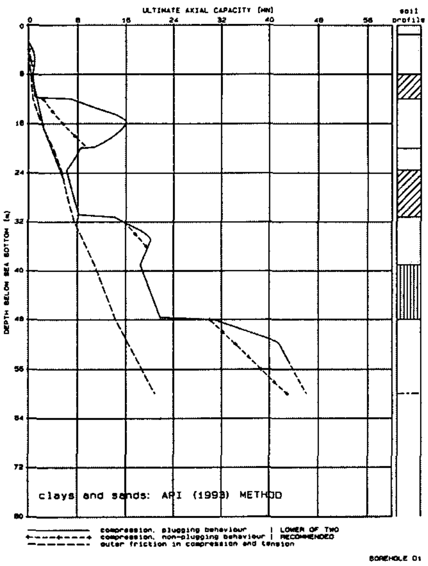
Pile penetration depth was selected to provide factor of safety of at least 2.0 with respect to normal operating loads, and at least 1.5 with respect to maximum design storm loads. These factors of safety should be applied to the design compressive and tensile loads. Axial load-pile movement analyses may be performed using a computer solution based on a method developed by Reese (1964) or Matlock et al (1976). These methods treat the pile as a series of discrete elements represented by linear springs that are acted upon by the nonlinear springs representing the soil. This type of analysis is referred to as a t-z analysis. In addition, the relationship between the pile tip load and pile tip movement (Q-z) was also analyzed.



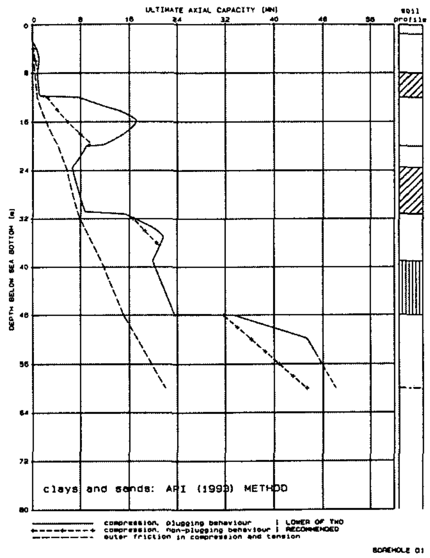
(a) 56 inch(1422.4 mm) diameter



(b) 60 inch(1524 mm) diameter



(c) 72 inch(1828.8 mm) diameter



(d) 76 inch(1930.4 mm) diameter

Fig. 7 Ultimate Axial Pile Capacity for 56inch(1422.4mm), 60inch(1524mm), 72inch(1828.8mm) and 76inch(1930.4mm) Diameter Pipe Piles with Depth

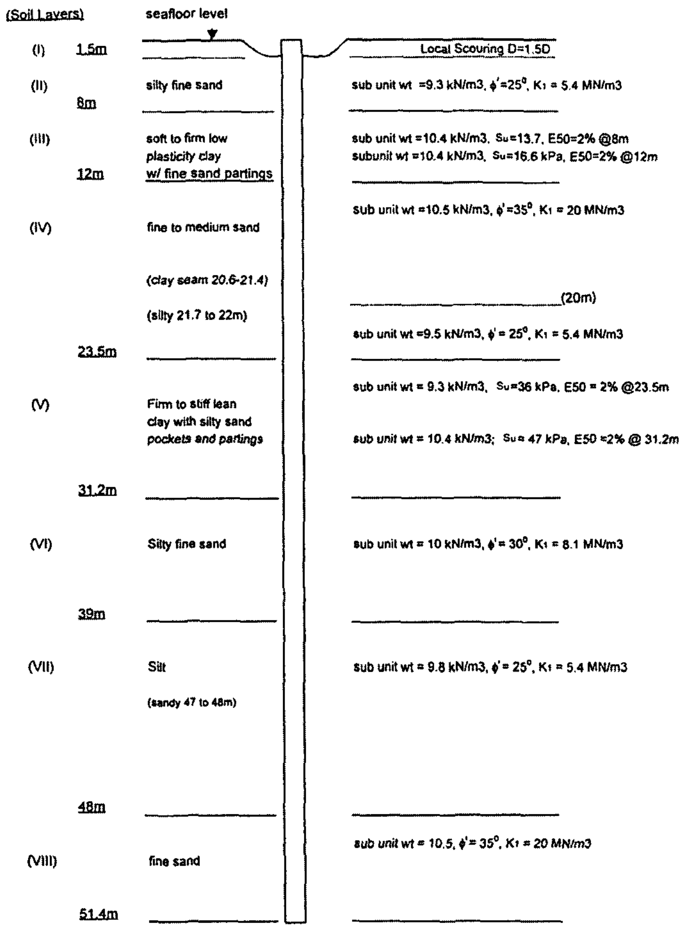


Fig. 8 The Stratigraphy and Soil Parameters Used for p-y Data

To provide the input information for the axial load movement analyses, procedures provided by API RP 2A-WSD (1993) were used in this study to develop the t-z and Q-z data. The results for individual pile size, 56, 60, 72 and 76inch diameter are presented in Fig.7. In the t-z data development, 20percent degradation in the soil-pile adhesion beyond peak values for static loading conditions was used. The lateral soil resistance-pile deflection(p-y) characteristics of the soils at the soil boring location were developed for each of the 56inch(1422.4mm), 60inch(1524mm), 72inch(1828.8mm) and 76inch(1930.4mm) diameter driven pipe piles. These data may be used in lateral load analyses of the proposed pile.

The static and cyclic p-y data were developed to 31m penetration using the procedures proposed by Matlock (1970) for clay and ONeill and Murchison (1983) for sands. The p-y data were developed by using the stratigraphy and parameters in Fig.8.

3.2 Scour Potential

The scour potential at a site depends on the pile size, grain-size distribution, relative density or consistency of the surficial soil deposits, current velocity, direction of flow and water depth. Based on the results of the field investigations and those of the laboratory tests, the surficial deposits consist of loose to medium dense fine sand. Considering the environmental conditions at the site and the characteristics of the surface sediments, scour potential and its effects were taken into consideration in the p-y data analysis. In developing the p-y data, effects of scouring around the pile at seafloor were taken into consideration. The depth of local scouring around the pile was taken to be 1.5 times the diameter of the pile, which can be assumed at normal wave and scouring regions.

3.3 Soil Resistance to Driving

To facilitate pile driveability study it is necessary to estimate the soil resistance to driving (SRD). SRD may be analyzed using the static method of analysis for axial pile capacity where the selected soil-pile parameters reflect the disturbance caused by pile driving and the build-up of excess pore water pressure. If the actual driving resistance mobilized by the soil is greater than the driving resistance that can be overcome by a particular hammer, then piles driven with that hammer may experience hard driving and perhaps refusal. For the present site condition, the SRD estimation may be based on pile driving in unplugged condition. The resistance to driving that may be mobilized by the soil at a given pile penetration may be estimated by the following typical criteria. During driving, it will be necessary to interrupt driving operations in order to make pile add-on or change hammers. The interruptions of driving operations may last six to eight hours. Delays for several days or more may result from bad weather or equipment breakdown. During this time, many clays gain strength as excess pore pressure dissipates and the soil particles reorient themselves. This phenomenon is commonly referred to as set-up. A similar phenomenon may also occur in fine-grained granular deposits. Upon re-driving the piles after some set-up has occurred, increased blow-counts may be experienced. Due to set-up, the soil resistance to driving at the beginning of the re-driving may increase to the point of refusal. It is suggested that the driving program should be planned so as to reduce the number and duration of delays.

4. Criteria For Axial Pile Load Analysis

The static method of computing axial pile capacity was used to compute ultimate compressive and tensile capacities of pipe piles installed to a given penetration. In this method, the ultimate compressive capacity (Q) for a given penetration is taken as the sum of the skin friction on the pile wall (Q_s) and the end bearing on the pile tip, (Q_p) so that:

$$Q = Q_s + Q_p = fA_s + qA_p \qquad (1)$$

Where A_s and A_p represent, respectively, the embedded pile surface area and pile end area;
f and q represent, respectively, the unit skin and unit end bearing resistance.

Procedures used to compute values of f and q are discussed in the following paragraphs. When computing ultimate tensile capacity, the end bearing term in the above equation is neglected.

4.1 Unit Skin Friction(f)

4.1.1 Cohesive Soils

Computation of unit skin friction for pipe pile driven in cohesive soils follows the API RP 2A-WSD (1993) method. According to the API (1993) method the unit skin friction (f) may be expressed as:

$$f = \alpha s_u \tag{2}$$

where α = dimensionless factor; and
s_u = undrained shear strength of the soil at the point in question.

The reduction factor can be computed by:

$$\alpha = 0.5 \Psi^{-0.5} \quad \text{for} \quad \Psi \leq 1.0$$
$$\alpha = 0.5 \Psi^{-0.25} \quad \text{for} \quad \Psi > 1.0$$

With the contraint that $\alpha \leq 1.0$,

Where $\Psi = s_u / \sigma_v'$ for the point in question
σ_v' = effective vertical stress at the point in question.

4.1.2 Granular Soils (siliceous)

The procedure recommended by API RP 2A-WSD (1993) is used to determine unit skin friction for pile driven in siliceous granular soils. Unit skin friction (f) is a function of the lateral earth pressure against the pile, and the angle of friction between the pile and the soil. The relationship can be expressed as follows:

$$F = K \sigma_v' \tan \delta \tag{3}$$

where K = coefficient of lateral earth pressure,
σ_v' = effective vertical stress at the point in question;
δ = friction angle between the soil and the pile surface

API RP 2A-WSD (1993) also presents recommended values for δ, N_q and specifies limiting values of unit skin friction. These recommended values are tabulated in API(1993). The values used for this study were selected from the recommended values on the basis of the field visual inspections, results of grain size distribution and the result of the shear box test.

4.2 Unit End Bearing Resistance (q)

4.2.1 Cohesive Soils

Procedure recommended by API was used to determine unit end bearing in clays. Unit end bearing (q) in clays can be estimated by the following equation:

$$q = 9 s_u \tag{4}$$

Where s_u = undrained shear strength of the soils at the point in question.

4.2.2 Granular Soils (siliceous)

Unit end bearing (q) in siliceous granular soils can be computed using the expression:

$$q = \sigma_v' N_q \tag{5}$$

Where ; σ_v' = effective vertical stress at the point in question; and
N_q = dimensionless bearing capacity factor that is a function of ϕ', effective friction angle of the soil.

Recommended bearing capacity factors (N_q) for granular soils composed primarily of silica given by API RP 2A-WSD (1993) and used in this study are tablated in Table 1.

Table 1. Design Parameters for Cohesionless Siliceous Soils

Density	Soil Description	Soil-Pile Friction Angle (Degree)	Limiting Skin Friction Values(f_L) kPa	N_q	Limiting End Bearing Values(Mpa)
Very Loose Loose Medium	Sand Sand-Silt** Silt	15	47.8	8	1.9
Loose Medium Dense	Sand Sand-Silt** Silt	20	67	12	2.9
Medium Dense	Sand Sand-Silt**	25	81.3	20	4.8
Dense Very Dense	Sand Sand-Silt**	30	95.7	40	9.6
Dense Very Dense	Gravel Sand	35	114.8	50	12

** Sand-Silt include those soils with significant fraction of both sand and silt.

4.3.3 Equivalent Unit End Bearing

For open-ended driven pipe piles, this study assumed that the end bearing was limited to the frictional resistance of a soil plug developed inside the pile. The unit skin friction on the inside of the pile was assumed equal to that on the outside of the pile. Any influence of the driving shoes on the internal skin friction was neglected in this study.

5. Criteria For Static Axial Load Transfer Data

The axial load movement analysis for the study of axial pile performance employs axial load transfer curves. These curves describe axial pile shear transfer as a function of axial soil-pile movement (t-z) in order to model the axial support provided by the soil along the side of the pile. Additional curves (Q-z) are used to model the tip end bearing-tip movement response. A brief discussion of the methods for constructing t-z and Q-z curves for piles driven in cohesive and granular soils is presented in the following sections. The procedures were also outlined in API RP 2A-WSD (1993).

Various empirical and theoretical methods are available for developing curves for axial load transfer and pile displacement (t-z) data. These methods include those proposed by Coyle and Reese, 1996; Coyle and Sulaiman, 1967; Vijayvergiya, 1997; Kraft et al, 1981; and Bogard and Matlock, 1990. On the basis of available pile load test data, the residual adhesive ration, degradation ratio in cohesive sediments is generally between 0.7 and 0.9 which depends on the factors such as soil stress-strain behavior, stress history, pile installation method, and pile load sequence.

In this study a degradation ratio value of 0.8 was assumed on the basis of previous experience. Typical t-z curves and the recommended methods of curves development for clay and sand are presented in Fig. 9. Please note that variables in Fig. 9 can be described as: z is local pile deflection in mm, D is pile diameter in mm, t is mobilized soil pile adhesion in t/m^2, t_{max} is maximum soil pile adhesion or unit skin friction capacity computed, and t_{res} is residual soil pile adhesion in t/m^2.

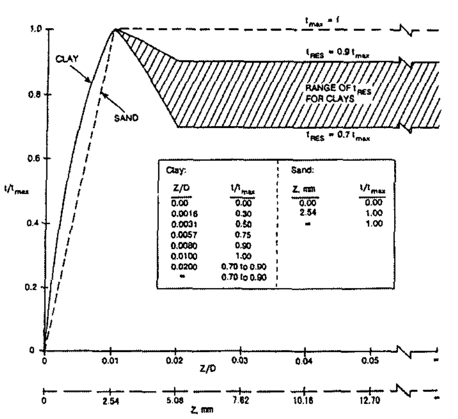


Fig. 9 Typical Axial Pile Load and Pile Displacement (t-z) Curves for Clay and Sand

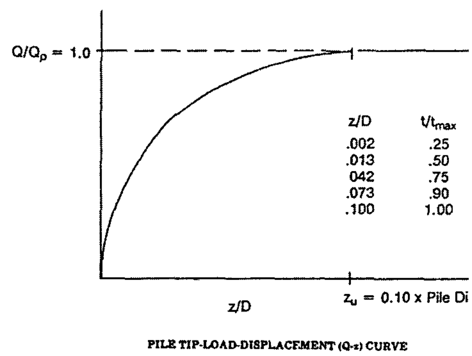


Fig. 10 Tip Resistance Curves used for Load Transfer Data

5.1 Tip Load Transfer (Q-z) Data

For siliceous sands and clays, large pile tip movements, as high as 10 percent of the pile diameter are required to mobilize the full end bearing resistance. The tip load resistance curves used for this study are presented in Fig. 10. The variables in Fig. 10 are defined as: z is axial tip deflection, D is pile diameter, Q is mobilized end capacity and Q_p is total end bearing capacity computed according to API code(API, 1993).

6. Criteria For Soil Resistance-Pile Deflection Data

6.1 Cohesive Soils

Soil resistance-pile deflection (p-y) data for cohesive soils were developed using the procedures outlined by Matlock (1970) and API RP 2A-WSA (1993) as shown in Fig. 11 for soft clay subjected to static and cyclic loads. Input parameters for the computation includes shear strength, submerged unit weight and dimensionless empirical constant, J. Interpreted shear strength and submerged unit weights used in this study are presented in Fig. 4. Also the interpreted strain value at one-half the maximum deviator stress (ε_{50}) was plotted in Fig. 5. These strain values were selected on the basis of the data from laboratory tests and the recommendations by Matlock (1970). The constant, J has values ranging from 0.25 to 0.5. For this analysis a J value of 0.5 was used.

6.2 Granular Soils

Soil resistance-pile deflection (p-y) data for siliceous granular soils were developed using procedures outlined by O'Neill and Murchinson (1983), and API RP 2A-WSD (1993) as shown in Fig.12 for sands subjected to static or cyclic loading. The procedures are the same for both types of loading. Input parameters include effective vertical stress (σ_v'); effective angle of internal friction (ϕ'); initial modulus of horizontal sub-grade reaction, k_1; and lateral soil pressure coefficient at rest, K_o. Values of σ_v' and ϕ' were selected on the basis of the results of laboratory tests. The k_1 values were selected on the basis of the σ_v' value as well as the recommendations of API. A K_o value of 0.4 was assumed for the analysis

7. Conclusionss

Design recommendations for soil parameters were proposed for pile design analyses based on an assessment of field investigation data as well as the laboratory test data. Pile design data presented in this paper for 56inch(1422.4mm), 60inch(1524mm), 72inch(1828.8mm) and 76inch(1930.4mm) diameters open-ended driven pipe piles include: a) ultimate axial pile capacity curves (compression and tension), b) axial load transfer (t-z and Q-z) data and c) static and cyclic (wave loading) lateral soil resistance-pile deflection (p-y) data.

The axial pile capacity curves were developed to 60m below the seafloor level. Since the soil boring was carried out to 51.4m penetration only, it has been assumed that stratum VIII continues to a depth in excess of 80m. The axial and lateral pile design data were developed using methods and recommendations presented in API RP 2A-WSD (1993). Recommendations for pile foundation design contained in this paper were developed based on the assumption that soil conditions as revealed by the soil boring are continuous throughout the general area of the proposed platform. Since information on high resolution geophysical survey on the investigated area was not available, no evaluation of possible stratigraphic changes, faulting or geologic conditions that might influence foundation design at the location was performed.

Acknowledgement

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References

1. American Petroleum Institute (1993), Recommended Practice for Planning, Design, and Construction Fixed Offshore Platform-Working Stress Design, API Recommended Practice 2A-WSD(RP 2A-WSD), 20th ed., API, Washington, D.C.
2. American Society for Testing and Materials (1993), "Soil and Rock; Dimension Stone; Geosynthetics," Annual Book of ASTM Standards, Vol. 04.08, ASTM, Philadephia.
3. Young, A.G. and Focht, J. A., jr. (1981). "Subsurface Hazards Affect Mobile Jack-up Rig Operations," Soundings, McClleland Engineers, Inc., Houston, Vol. 3, No. 2, pp. 4-9.
4. Matlock, H. (1970), "Correlation for Design of Laterally Loaded Piles in Soft Clay," Proceedings, Second Offshore Technology Conference. Houston, Vol. 1, pp577-594.
5. O'Neill, M.W., and Murchinson, J.M. (1983), An Evaluation of p-y Relationships in Sands, Report PRAC 82-41-1, Prepared for the American Petroleum Institute, Houston, May.
6. Sullivan, R.A., and Ehlers, C.J. (1972), "Practical Planning for Driving Offshore Pipe Piles," Proceedings, Fourth Offshore Technology Conference, Houston, Vol. 1, pp. 805-822.
7. Terzaghi, K., and Peck, R.B (1967). Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley & Sons, p.729.

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