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Yarcheng 13-1 고정식 해양구조물의 파일 지지력에 관한 연구

An Investigation on Long Offshore Pile Resistances of Yarcheng 13-1 Process Fixed Offshore Platform

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개요 : 고정식 해양구조물의 파일 설계와 관련하여 파일 지지력 선정을 위한 API 기준과 그 동안 전문가들이 제안한 방법 중 Offshore long pipe pile에 적용성이 인정된 방법들의 특성조사, 그리고 각각의 제안방법들의 적용성을 비교검토하기 위하여 기수행한 Yarcheng 13-1 Process Platform에 각각의 방법들을 적용하고 그에 따른 파일 길이와 제작 및 시공 비용과의 상관관계를 검토하였다.
기준의 적용에 따라 파일의 길이, 제작 및 시공 비용에 많은 차이가 있었고, 본 공사의 파일설계는 정규압밀 점토가 대부분인 현장토질 특성을 너무 과소평가하여 Spec. 기준보다 과다하게 설계되었음을 알 수 있었고, API 1993 기준, Randolph & Murphy(1985), Semple & Ridgen(1984)에 의해 제안된 방법들이 비교적 본 현장의 토질특성에 적절하다고 판단되었다.

주요어 : Offshore, Open-ended driven pipe Pile, API RP2A, Friction pile, Soil plug

1.0 Introduction

Most offshore platforms are supported by deep foundations that consist of long, open-ended, driven pipe piles or drilled and grouted piles. The pile foundation must be adequately designed to carry the static, cyclic, and transient loads.

Thus, foundation design/analyses are required to confirm that the pile members are having sufficient axial and lateral capacity during extreme loading conditions. There is a close relationship between design and construction/site condition including driving practices. The result of the design/analyses is the sizing of the pile member for diameter, penetration, and wall thickness to confirm that structure will produce safe and risk-free installation and performance.

The office-oriented pile design is to determine the pile diameter, wall thickness and penetration to provide adequate tensile and compressive capacities with an appropriate factor of safety(as ultimate capacity) because no rational analytical design method exists, which can capture the effects of all factors of significance for the axial resistance of pile and to check the driveability of pile.

Meantime, load test is most reliable to evaluate the ultimate bearing capacity of pile, however particularly on the sea, it is almost impossible due to the size and capacity of pile and requires considerable time and cost, which is not possible to consider in design phase. Another means of prediction of pile resistance in design phase is wave equation analysis method. but this dynamic method also can not give a reliable value of resistane as limited by applied soil strength and parameters.

Normally offshore long pile is to be designed as non-uniform pile.

In this paper, prediction and comparison of the penetrations as per the required capacity and their construction costs of open-ended driven pipe pile of the Yarcheng 13-1 Process fixed offshore platforms are to be presented in order to check the applicability of the proposed methods such as API RP2A(1991/1993), API RP2A(1986), Randolph & Murphy(1985), Semple & Ridgen(1984), Dennis & Olsen(1983), Beta Method(1973), and Lamda Method(1972).

2.0 Site and Soil Condition

The experimental site is located in the Yarcheng 13-1 natural gas field off Hainan Island in the South China Sea, People's Republic of China. Water depth of the site is about 80m. A 480-mile subsea pipeline extends from the Yarcheng field to Hong Hong where the gas is being used for electric power generation. Yarcheng 13-1 has an estimated 3 trillion cubic feet of recoverable reserves and a projected field life of 20 years.

This soil stratigraphy was disclosed on the boring logs and CPT logs. Detailed soil description that incorperate textural, color variations and inclusions are included on the boring log in Fig.2A. The geotechnical parameters were obtained from a rotary-cored borehole to a depth of 140m with UU triaxial tests, the in-situ vane tests and PCPT tests. The results of the in-situ vane tests and PCPT tests are in good agreement. The results indicate that the soils at the site are normally consolidated clay. Soil parameters and conditons for axial pile capacity are summarized in Fig. 2B.

3.0 Descrition of Pile Foundation

The piled foundation of the fixed offshore platform consists of eight(8) numbers of open-ended long steel pipes of 60"(1524mm)diameter and 1.50-2.625"(38-67mm) wall thickness. The penetrations were about 395ft(120m) below mudline. The maximum axial compression load from the jacket reaction for 100 years storm condition was 31 MN, consequently the required ultimate capacity of pile was 46.5 MN with the factor of safety of 1.5.

Specification requirement of the pile foundation is as per API RP2A, WSD(1991).

4.0 Review of Propo··d Methods and Parameters

In general, the ultimate capacity calculations of offshore open-ended pipe pile are based on the summation of the shaft friction and end-bearing components of resistance as follow:

$$Q_{ult} = \Sigma f_s A_s + \Sigma f_i A_i + q_p A_w$$

where, $f_s A_s$ = external unit friction * external shaft area,

$f_i A_i$ = internal unit friction * internal shaft area,

$q_p A_w$ = unit end-bearing * annulus end area.

If the accumulated internal friction exceeds the plug, end-bearing capacity and the pile capacity is given by:

$$Q_{ult} = \Sigma f_s A_s + q_p A_p$$

where, A_p = gross end-bearing area

Ultimate pile resistance in compression and in tension of driven pipe pile is determined as follows:

$$Q_{ult,c} = Q_1 + \mathrm{Inf}[\,Q_{p1},\, Q_{p2}\,] - W' \quad \text{(Compression)}$$

$$Q_{ult,t} = Q_1 + W'' \quad \text{(Tension)}$$

where, $Q_1 = \Sigma f_s A_s$, $Q_{p1} = q_p A_p$, $Q_{p2} = q_p A_w + \Sigma f_i A_i$

$W' = \Sigma A_w(\gamma_p - \gamma)\Delta L$; submerged, $W'' = \Sigma A_w[(\gamma_p - \gamma_w) + A_i \gamma']\Delta L$; buoyant + internal

γ = total unit weight of soil, γ_p = specific weight of steel(77 kN/ m^3),

γ_w = specific weight of water, γ' = buoyant unit weight of soil.

And, the allowable pile capacities(P_c and P_t)of the fixed offshore platform are obtained by applying specific safety factors(SF) of 2.0 and 1.5 for normal operating and extreme storm conditions, respectively or by multiplying pile resistance factor to the calculated ultimate capacities as per API RP 2A or Dnv.

The design of offshore pile in cohesive clay soils is based largely on the experience with onshore piles due to the lack of field pile test database.

The methods developed are empirical and subject to the limitations and uncertainties in the database. There is also a need to extend the database by conducting field pile test in soil types more relevant to offshore condition.

Piles driven into clay soils derive most of their capacity from skin friction resistance along the length of pile. Thus, many research efforts have been conducted to investigate theoretical and empirical methods for determining the unit skin friction. The unit skin friction, f_{ave}, was computed as a function of the undrained shear strength, C_u, of the soil by means of an empirical frictional coefficient α, as follows;

$$f_{ave} = \alpha C_u$$

The value of α was first deduced empirically from pile load tests by Peck(1958) and Tomlinson(1957). In recent years, several other investigators such as Randolph and Murphy(1985), Semple and Ridgen(1984), Dennis & Olsen(1983) have proposed various relationships between α and C_u. Generally Tomlinson method is not suitable for the offshore long pile, which is based on data for relatively short piles in the terrestrial environment. Tomlinson's low values of α presumably developed because of lateral deflections of pile during driving at shallow depths but such deflections would not occur offshore.

Another procedure proposed by Vijayvergiya and Focht(1972) correlates the total friction capacity of piles embedded in clay to both the undrained shear strength and the in-situ effective vertical stress. The average shaft friction(f_{ave}) on the embedded pile is computed from

$$f_{ave} = \lambda(\dot{\sigma}_v + 2\,C_u)$$

where, λ = a dimensionless coefficient

$\dot{\sigma}_v$ = mean effective vertical stress along the pile

C_u = mean undrained shear strength along the pile

The λ factor, which is a function of the pile penetration.

One of the earliest effective stress methods was proposed by Chandler(1968) that considered the skin friction between the soil and pile to be frictional for clay soils in the same way as piles develop their frictional capacity for free draining granular soils. Thus, the resulting skin friction is a function of the normal effective stress, σ_n, and a pile-soil friction angle, δ. The normal pile skin friction is related to the effective overburden stress, σ_v', by an earth pressure coefficient, K_o by

$$f = \sigma_n \tan\delta = K_o \sigma_v' \tan\delta = \beta\sigma_v'$$

where $\beta = K_o \tan\delta$

The value of K_o depends upon the past stress history of the soil and the method of pile installation.

Burland(1973) suggested that δ should be taken as the effective angle of internal friction, ϕ', for a remolded soil, and $K_o = 1 - \sin\phi'$ for normally consolidated clays. Thus, β would vary from 0.24 for ϕ' of 20^o to 0.29 for 30^o in a normally consolidated soil. His recommendations do not allow for stess changes that occur during pile installation to be included into the parameters used for design. For overconsolidated soil, Burland leaves the determination of K_o to the discretion of the designer.

Meyerhof(1976) later suggested that K_o should be about 1.5 times the earth pressure coefficient for driven piles in stiff clay and 0.5 times the earth pressure coefficient for bored piles. Thus, he backfigured β values in normally consolidated clays as 0.3 at shallow penetrations and decreasing at deeper penetrations to account for pile length effect. For overconsolidated clays, β may be approximated by the following expression;

$$\beta = 1.5(1 - \sin\phi')(OCR)^{0.5}\tan\phi'$$

where OCR is the overconsolidated ratio, as defined previously. Meyerhof does not propose a pile length adjustment for overconsolidated clays; however he does point out that the confidence level is less for overconsolidated clay because of the large scatter in back-calculated β values.

Subsequent research efforts included four generations of effective stress methods(1982). These methods attempted to correlate the frictional capacity of piles in clay to the changes in effective stress in the soil that occur as the initial free-field stress changes after pile installation, during and after consolidation, and during pile loading. Althrough these theoretical models provide useful insight into the factors that affect skin friction resistance in clays, the new generation of effective stress methods are not ready to use as a standard design method at this time.

In more recent years, investigators such as Randolph and Murph(1985) and Semple and Rigden (1984) concluded that unit skin friction not only depends on shear strength but also on the past stress history as reflected by the strength ratio, C_u/σ_v'. The API RP2A(1987-1993) combined the results of these two studies into equations as follows;

$$\alpha = 0.5/\psi^{0.5} \quad \text{when} \quad \psi \le 1.0 \quad \text{or}$$

$$\alpha = 0.5/\psi^{0.25} \quad \text{when} \quad \psi \ge 1.0$$

with the constraint that $\alpha \le 1.0$, and where, $\alpha =$ the adhesion factor

$\psi = C_u/\sigma_v'$, normally consolidated strength ratio for the depth of interest

$\acute{\sigma}_v$ = the effective overburden pressure at the depth of interest

None of these empirical procedures for computing the unit skin friction in clay soils consider a major uncertainity associated with the effect of pile length on the mobilized unit skin friction. Kraft (1982) suggests that three primary factors may contribute to the so-called "length effect".

For long piles, the reduction in shaft resistance increases with pile length as reported by Vijayvergiya and Focht(1972), Meyerhof(1976), Floate and Selnes(1977), Murff(1980), Kraft(1982), Semple and Rigden(1984), and Randolph and Murphy(1985).

All these procedures define how the static unit skin friction values should be adjusted to estimate the total length effect which results in mobilized friction ratio of less than 1.0 for long piles.

The total end bearing resistance of a pile driven into clay is a small percentage of the total ultimate capacity. The unit end bearing of a driven pile in clay is also related to the undrained shear strength, C_u, by a bearing capacity factor, N_c, as follows;

$$q = N_c C_u$$

For deep penetration piles, N_c is generally taken as 9, as recommended by Skempton(1951). The end bearing capacity is obtained by multifying the unit end bearing by the total pile end area when the open-ended pile develops a full soil plug.

The axial capacity of piles driven into cohesionless granular soils such as silts, sands, and gravels depends primarily upon the angle of internal friction, ϕ, and the effective overburden pressure, $\acute{\sigma}_v$.

Both the skin friction resistance and end bearing resistance will increase approximately proportionally with depth. Most design methods are based on empirical correlations to sampler blow count information or correlations to cone penetrometer data.

The unit skin friction in cohesiveless granular soils is generally computed from

$$f = K_o \acute{\sigma}_v \tan\delta$$

where K_o = the earth pressure coefficient

$\acute{\sigma}_v$ = the effective overburden pressure

δ = the friction angle between the soil and pile material

For open-ended driven piles with minimal soil displacement, K_o is generally taken as about 0.8 for both compression and tension. For closed-ended piles, K_o may increase up to 1.0 or greater.

The friction angle, δ is generally assumed to be 5^o less than the angle of internal friction of the soil. Design parameters commonly recommended in the API RP2A(1986-1993) for granular soils are varying texturally from silt to gravel with densities ranging from loose to very dense. When CPT data is available, unit skin friction can be taken as proposed by de Ruiter and Beringen(1979).

As previously described, the equation for unit skin friction, f will increase proportionally with depth; however, research by Vesic (1977) indicates that the rate of increase of f with depth gradually reduces to a point where some limiting value of skin friction, $f_{\lim}$ is reached.

The unit end bearing resistance of piles driven in granular soils is computed by

$$q = N_q \acute{\sigma}_v$$

in which N_q is a bearing capacity factor that is related to the textural and density characteristics of the soil. The work of Vesic(1977) also demonstrates that unit end bearing reaches a limiting value, $q_{\lim}$

The API RP2A recommendations also classify limiting skin friction and unit end-bearing values in cohesionless siliceous soils according to the relative density and soil-pile friction angle, δ.

The limiting values advocated for siliceous sands should absolutely not be applied to the case of limestone/calcareous sands, for which relevant informations proposed by Nauroy et al(1988) and Nauroy & Le Tirant(1985).

5.0 Pile Resistance Prediction

Following methods such as API RP2A(1991/1993), API RP2A(1986), Randolph & Murphy(1985), Semple & Ridgen(1984), Dennis & Olsen(1983), Beta (1973), and Lamda(1972) for clay and API RP2A(1991/1993) for sand had been used .

Each procedure has been covered more thoroughly in several appendices in order to avoid exposing to excessive detail in this paper.

Soil parameters and limit values for the pile capacity have been given in Fig. 2B.

The pile resistance/capacity in compression of 60" pile, calculated using the above-mentioned procedures, is given in Table 5A.

As mentioned, the required ultimate capacity of pile was 46.5MN with the safety factor of 1.5 for maximum design storm condition in accodance with API RP2A(1991) recommendations.

The ultimate capacities according to the above-mentioned procedures and the pile penetrations relating to the required ultimate resistance, 46.5MN, have been listed in Table 5A and Fig.5B.

Table 5A: Summary of the pile resistances/penetrations of 60" pipe pile

Depth (m)	API 1993 (KN)	API 1986 (KN)	R/M 1985 (KN)	S/R 1984 (KN)	D/O 1983 (KN)	Beta (KN)	Lambda (KN)
0-16.4	2430	2430	2430	2430	2430	2430	2430
16.4/30.0	5401	4511	5123	5338	3878	5446	5335
30.0/39.0	9602	7047	9025	9781	7956	9019	8905
39.0/65.0	25754	15944	23890	26869	21785	19049	21446
65.0/69.0	30166	20354	28299	31278	26194	23458	25855
69.0/86.0	39388	23539	37446	40500	32530	29123	32715
86.0/98.0	49236	28610	47511	49657	39684	36555	41012
98.0/104	54823	31551	53193	54678	43551	40767	45711
104/110	56638	33366	55008	56492	45365	42581	47525
110/137	89425	50070	86786	83581	65093	64759	73111
137/147	93833	54478	91194	87989	69501	69167	77519

6.0 Variation of the Required Pile Penetrations

The variations of the pile penetrations relating to the required ultimate resistance, 46.5MN, have been listed in Table 6A.

Actual penetration of 60" open-ended long steel piles(8 Nos) of Yarcheng 13-1 Process Platform was 120m into the soil from sea bottom.

Comparatively, actual penetration, 120m, is much higher than the predicted pile penetrations.

Table 6A: Variation of pile penetrations

Depth (m)	API 1993 (KN)	API 1986 (KN)	R/M 1985 (KN)	S/R 1984 (KN)	D/O 1983 (KN)	Beta (KN)	Lambda (KN)
Required penetration(m)	95m	132m	97m	94m	112m	115m	107m
Deviation (m)	-25m	+12m	-23m	-26m	-8m	-5m	-13m

7.0 Cost Comparison

The comparisons of the construction costs as a function of pile penetration depth, are shown in Table 7A. As can be seen, the deviation of construction costs are also much higher.

Table 7A: Comparison of the construction costs

Depth (m)	API 1993	API 1986	R/M 1985	S/R 1984	D/O 1983	Beta	Lambda
Length Deviation (m)	-25m	+12m	-23m	-26m	-8m	-5m	-13m
Weight Deviation (tons)	-286tons	+137tons	-263tons	-297tons	-92tons	-57tons	-149tons
Fabricatioe Cost (US$)	-286,000	+137,000	-263,000	-297,000	-92,000	-57,000	-149,000
Installation Cost (US$)	-160,000	+80,000	-160,000	-160,000	-80,000	-80,000	-80,000
Total Cost (US$)	-446,000	+217,000	-423,000	-457,000	-172,000	-137,000	-229,000

Notes:1. Pile fabrication cost: US$1000/ton (1995 base)

2. Pile construction/installation cost: US$40000/day,(1995, 1200ton offshore crane barge daily rate base)

8.0 Conclusion and Recommendations

The paper discusses the office-oriented offshore pile design procedures affecting the pile penetration and their construction costs.

The API 1986 method highly underestimates the skin friction of the soils. On the other hand, the API 1993, Randolph & Murphy(1985), and Semple & Ridgen(1984) yield predictions of capacity which are very similar at all depths. The soils of the Yarcheng 13-1 field are normally consolidated clay.

Accordingly, these three(3) procedures can be considered as relatively reliable in this type of normally consolidated soil.

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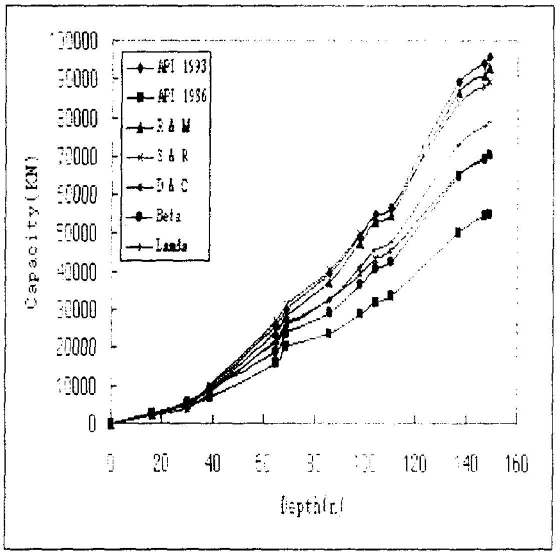

Fig. 5B Summary of the pile resistance curves

Fig. 2b Soil Parameters and conditions

Main pile analysis, Yancheng [illegible] Platform

layer number	depth [m]	soil type	dens	δ [deg]	c_u [kPa] top	c_u [kPa] bot	sb. wt [KN/m3]	σ_v' KPa	Ψ (c_u/σ_v')	N_q [-]	f_{lim} [kPa]	q_{lim} [MPa]
	0.0							0.0				
1		sand	[illegible]	15.0			10.0	7.7		8.0	47.9	1.9
	[illegible]							15.3				
[illegible]		sand	[illegible]	15.0			10.0	81.5		8.0	47.9	1.9
	16.4							147.7				
[illegible]		clay	[illegible]		43.1	[illegible]	7.5	209.0	0.[illegible]			
	30.0							270.2				
[illegible]		clay	[illegible]		[illegible]	[illegible]	8.0	310.7	0.3			
	39.0							351.3				
4		clay	[illegible]		[illegible]	[illegible]	8.5	468.3	0.[illegible]			
	65.0							585.4				
5		sand	m	20.0			9.2	603.5		[illegible]	[illegible]	2.9
	[illegible]							621.5				
6		clay	[illegible]		143.6	[illegible]	9.2	698.0	0.2			
	[illegible]							774.6				
7		clay	[illegible]		152.[illegible]	178.5	9.6	828.6	0.[illegible]			
	[illegible]							882.7				
8		clay	vd		[illegible]	[illegible]	9.0	909.7	0.[illegible]			
	104.0							936.7				
9		sand	[illegible]	[illegible]			9.0	963.7		[illegible]	47.9	1.9
	130.0							990.7				
10		clay	vsd		[illegible]	[illegible]	9.0	1112.3	0.[illegible]			
	137.0							1233.9				
11		sand	m	[illegible]			9.0	1279.0		[illegible]	67.0	2.9
	147.0							1324.0				
12		clay	vsd		277.7	277.7	9.0	1333.0	0.2			
	149.0							1342.0				

Fig. 2B Soil paramaters and limit values for the pile capacity

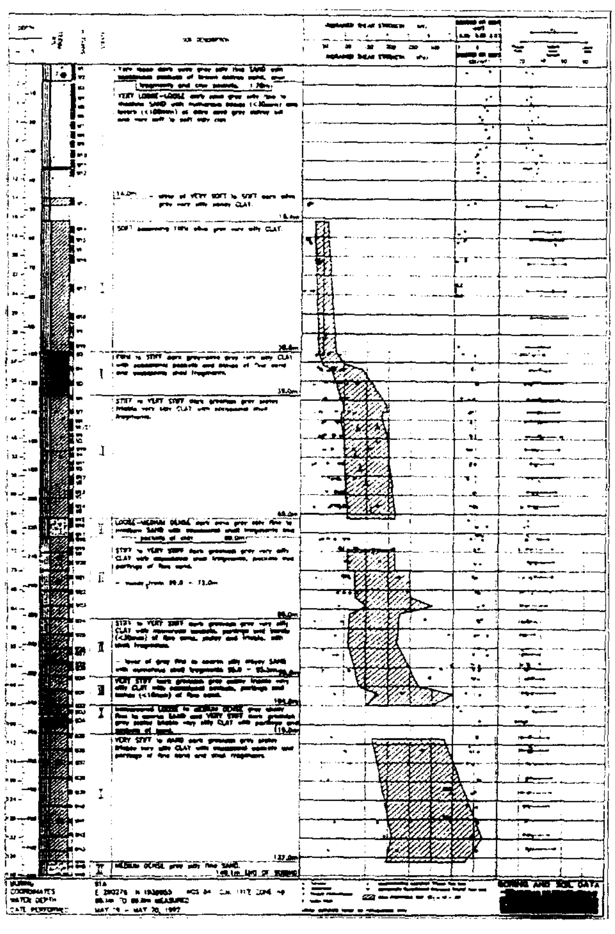

Fig.2A The boring logs