Pipe piles under mooring forces

Pipe en vertu de pieux d'amarrage forces

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ABSTRACT
Floating Production Storage Offloading (FPSO) Vessels are widely used in offshore oil and gas industry. In offshore Atlantic Canada, mooring piles driven in the seabed, in water depths ranging from 80 to 200 m, are used to moor these FPSOs. These mooring piles are subjected to oblique pull forces. In this paper, a 3D finite element method has been used to study the behaviour of steel pipe piles in saturated sand under mooring forces. The main objective of the present study is to check the validity of the available theoretical models in the literature. It has been found that most of the previous theoretical models should be modified to consider the prototype behaviour.

RÉSUMÉ
Flottante de production, de stockage de déchargement (FPSO) Les bateaux sont largement utilisés dans les gisements offshore de pétrole et du gaz. Offshore dans le Canada atlantique, pieux d'amarrage conduit dans les fonds marins, dans les profondeurs d'eau allant de 80 à 200 m, sont utilisés pour amarrer ces FPSO. Ces pieux d'amarrage sont soumis à des forces de traction oblique. Dans ce document, un 3D méthode des éléments finis a été utilisée pour étudier le comportement des piles de tuyaux en acier dans le sable saturé à l'amarrage des forces. L'objectif principal de la présente étude est de vérifier la validité des modèles théoriques disponibles dans la littérature. Il a été constaté que la plupart des précédents modèles théoriques devraient être modifiés pour examiner le comportement prototype.

Keywords : offshore, pile, mooring force, sand.

1 INTRODUCTION

Floating Production Storage Offloading vessels (FPSOs) are widely used in offshore oil and gas industry as an alternative to fixed production platforms in harsh environments reach at the Grand Banks, in water depths ranging from 80 to 200 m. Many FPSOs are keeping position using seafloor moorings which are commonly secured using pile anchors as shown in Fig.1. Correctly designed pile anchors should transfer the environmental loads on the floating platforms to the seabed safely. In-service, these anchors or mooring piles are subjected to a wide range of monotonic and cyclic lateral to oblique pull forces. The large cyclic forces applied during extreme storm will tend to govern the design. As reported by Bhattacharya (2007), the design of these mooring piles has not been codified as jacket piles which are widely used for offshore structures. Also, both piles are different in geometry and applied loads. While jacket piles are long/flexible, fixed-head and axially loaded piles (compression/tension), mooring piles are shorter/close to rigid, free-head and incline loaded piles. Therefore, the design of these mooring piles should not be the same as the jacket piles and extensive need to develop an accepted design method for this type of piles should be considered.

There is relatively limited experimental information on mooring piles or piles subjected to oblique pull loads. Some of the existing theoretical models are semi-empirical based on 1-g experimental tests as Yoshimi (1964), Broms (1965), Das et al. (1976), Chattopadhyay and Pise (1986), Ismael (1989), and Jamnejad and Hesar (1995). As indicated by Altaee & Fellenius (1994), the dilation of the sand occurring at low confining stress (at shallow depth) increases the lateral soil stress against the pile. So, doing a test even in the field using a small scale pile; as conducted by Leshukov (1975), and Ismael (1989) will only eliminate the boundary conditions problem in the laboratory test, but the physical modelling issue will not be controlled and therefore their results cannot correctly reproduce the real behaviour of piles under mooring forces for sandy soil. Other models are based on the net uplift and the ultimate lateral capacity of the pile, whichever is smaller, as reported by Poulos and Davis (1980) and so neglected the interaction between horizontal and vertical pull forces on the pile.

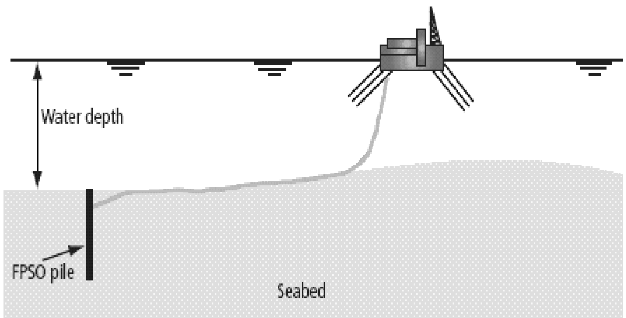

Figure 1. Schematic diagram of an FPSO and Anchoring System (after Bhattacharya 2007).

In this paper, finite element method has been used to study the behaviour of steel pipe piles in saturated sand under mooring forces. A 3D model was established at the prototype scale under different conditions of loading considering the soil-pile interaction behaviour. The main objective of the present study is to check the validity of the available theoretical models under static mooring forces. The calculated pile capacity by the finite element will be compared with the equation suggested by Das, et al. (1976):

$$\frac{P_\theta \cos\theta}{P_u} + \left(\frac{P_\theta \sin\theta}{P_L}\right)^2 = 1 \qquad (1)$$

where, P_θ = the pile capacity under mooring force with inclination angle θ to horizontal, P_u = the ultimate uplift capacity of the pile, and P_L = is the ultimate lateral capacity of the pile. Also, the results will be compared with the equation suggested by Chattopadhyay and Pise (1986):

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$$\frac{P_\theta}{P_u} = \cos^2\theta \exp\left(-\frac{1-\theta/_{90}}{1+\theta/_{90}}\right) + \frac{\sin\theta}{\alpha}\exp\left(-\frac{1-(90-\theta)/45}{1+(90-\theta)/45}\right) \quad (2)$$

where, $\alpha = P_u / P_L$.

2 SOIL CONDITION AT THE GRAND BANK

The soil conditions of the western location of Hibernia field in the Grand Bank have been selected for the analysis in this paper. The detailed soil characteristics are given by Thompson & Long (1989). Dense Sand was dominant from the sea floor to 50-60 m and hard cohesive soils alternating with layers of sand and silty sand were dominant below that. As the pile length studied in this paper is 30 m, the dense sand layer properties of 50 m depth were used in the analysis.

3 NUMERICAL ANALYSIS

Numerical analysis was carried out using the ABAQUS 6.7 finite element analysis program (Hibbitt et al. 1998). The finite element mesh used in the analysis is shown in Figure 2. The elements used are 8-node continuum elements with porous properties for those elements modelling the soil. Due to the symmetric loading condition only a half-cylinder representing the soil and the pile was considered. The elements are biased towards the pile in order to give most data in the region of greatest interest, i.e. close to the pile. Because there is no full scale or centrifuge test available in the literature, sensitivity analysis has been done to examine different mesh geometry. The one that used in the analysis was the one of less time processing and with results close to the one of finer mesh and more time processing. The limits of the mesh were at a diameter of 40 m which is 20 times the pile diameter and 50 m height, so the soil extends under the pile 5 times of the pile diameter.

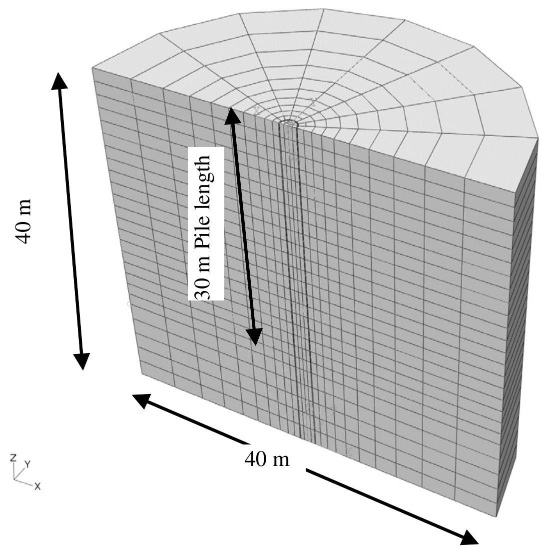

Figure 2. Finite Element Mesh for 2 m diameter and 30 m length steel pipe pile.

Table 1. Sand parameters used in the FE analysis.

Soil Parameter	Value
Dry unit weight, γ (kg/m^3)	1800
Young's modulus, E (kN/m^2)	74000
Poisson's ratio, ν	0.3
Angle of internal friction, φ	38°
Critical state friction angle, φ_{cv}	31°
Dilation angle, ψ	8.47°
Cohesion, c' (kN/m^2)	1.0

Steel pipe pile of 2 m diameter, 0.05 m wall thickness, and length to diameter ratio of 15 has been used in the analysis. The dimensions of this pile have been selected based on the in-service mooring piles at the Grand bank (personal communication with Husky Energy). The material behaviour of the pile was assumed to be linear elastic with the parameters; Young's modulus (E) = 2.1x10^8 kN/m^2 and Poisson's ratio (ν) = 0.2 for steel. The sand has been modelled as an elasto-plastic material with Mohr-Coulomb failure criterion. An average value of the angle of internal friction has been used in the analysis as Thompson & Long (1989) gave a variable distribution that decreases with depth up to 25 m depth and then be constant. Young's modulus has been calculated from the given value of the bulk modulus by Thompson & Long (1989) and an assumed Poisson's ratio of 0.3 for dense sand and considered to be constant with depth. The sand properties that have been used in the finite element analysis are given in Table 1. The dilation angle, ψ has been calculated based on Rowe's (1962) relation:

$$\sin\psi = \frac{\sin\varphi - \sin\varphi_{cv}}{1 - \sin\varphi\sin\varphi_{cv}} \quad (3)$$

where φ is soil friction angle, φ_{cv} is soil critical state friction angle.

As for driven piles, the soil pile interaction has been modelled using contact elements. The shear stress between the surfaces in contact was limited by a maximum value $\tau_{max} = \mu p$, where p is the normal effective contact pressure, and μ is the friction coefficient. A value of tan (0.6 φ) was taken for μ, as suggested by Popescu & Nobahar (2003).

In the finite element analysis, first step was the geostatic step for the soil to apply the soil gravity. In the next step the pile and the contact elements have been activated and a prescribed displacement has been applied at the top side node of the pile at the symmetry plan. The prescribed displacement has been applied with different angles; θ to horizontal; 0°, 30°, 45°, 60°, 90° to simulate the pile under mooring conditions. The angles 0° and 90° are not the case of mooring conditions; however they have been studied to get the ultimate lateral and pullout pile capacity.

4 RESULTS

4.1 *Load-displacement curves*

Figures 3 and 4 show the load-displacement curves of the horizontal and vertical components for the different inclination load angles. In Figure 3, the horizontal load component versus the horizontal displacement component is plotted. It can be seen that all curves have the same initial stiffness up to a certain load level after which the curves deviate from the curve of pure horizontal load; θ = 0°. As the load inclination angle increases, the stiffness of the curve decreases at a smaller horizontal displacement. This can be expected, as the ultimate lateral capacity of this pile is larger than the ultimate uplift capacity. By increasing the load inclination angle to horizontal, the vertical load component will gradually decrease the horizontal pile stiffness.

However, to better understand this behaviour, we can see Figure 4. The vertical load component versus the vertical displacement component is plotted. It can be seen that the initial stiffness of the load-displacement curves decreases slightly by increasing the load inclination angle to horizontal. Also, the stiffness for all curves start to decrease at certain level of load which is close to the ultimate uplift capacity of the pile

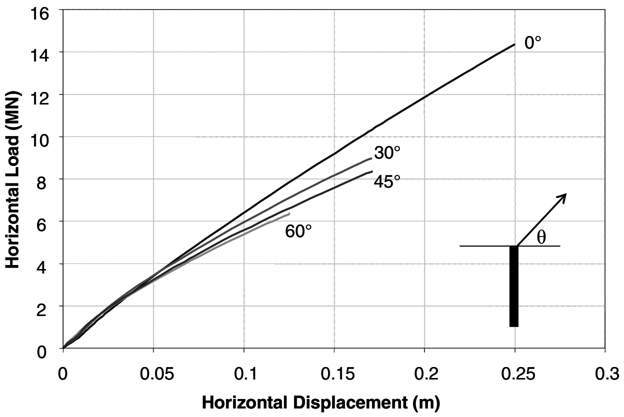

Figure 3. Horizontal load versus horizontal displacement curves at the pile head for different inclination load angles.

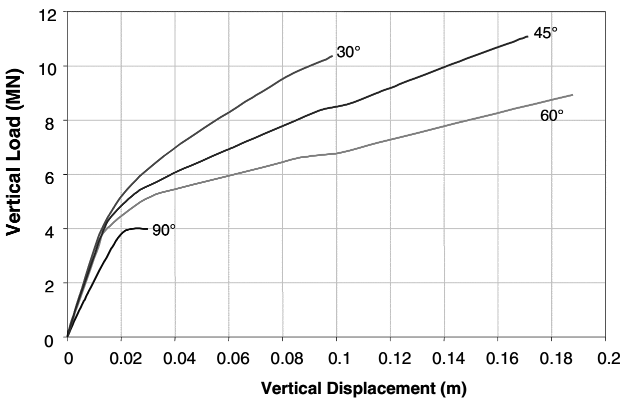

Figure 4. Vertical load versus vertical displacement curves at pile head for different inclination load angles.

as shown from the pure vertical loading curve. It can be concluded that the ultimate lateral capacity of this pile controls the initial loading stiffness of the pile, however, as much pull (10-15 mm) progresses the uplift capacity control the loading stiffness of the pile. However, more load inclination angles with small increments need to be studied to find the critical inclination angle of the load at which the failure changes from axial failure to lateral failure.

4.2 *Ultimate pile capacity*

Figure 5 shows the total load-displacement curves for the different load inclination angles. For the curves of θ = 30˚, 45˚, 60˚, and 90˚, the failure load can be easily picked by drawing the tangent to the initial and end portion of the curve. The intersecting point of the two tangents will give the failure load. However, the curve of θ = 0˚ (horizontal load) is flat curve and the ultimate capacity has been selected at 10% of the pile diameter, as described by Hesar (1991).

The ultimate uplift and lateral capacity obtained by the finite element model of the pile have been used in the recommended equations (1) and (2) to calculate the capacity of the pile under mooring force of angles θ = 30˚, 45˚, and 60˚. Table 2 shows the capacity values obtained from the finite element model and calculated from the mentioned equations. It can be seen that the calculated ultimate capacities by equation (1) are much closer to the predicted one by the finite element than those by equation (2). The reason of that much difference between the two equations in the estimated ultimate capacity is that what mentioned by Altaee & Fellenius (1994). Both equations are based on 1g test results. Because of the nonlinear stress-strain behaviour and the dependence of behaviour on initial level of

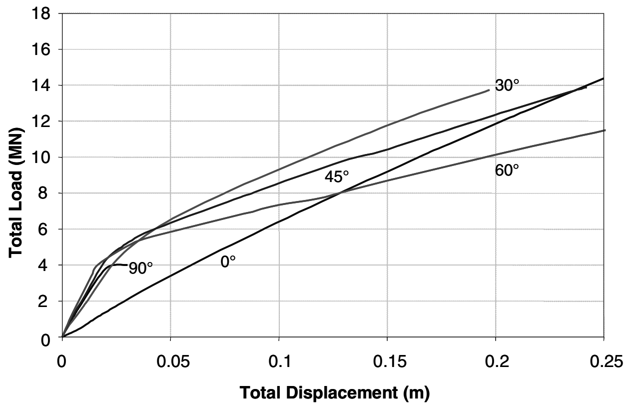

Figure 5. Total load versus total displacement curves at pile head for different inclination load angles.

Table 2. Ultimate pile capacity (in MN) calculated and predicted from different methods.

The Method	Load inclination angle θ				
	0˚	30˚	45˚	60˚	90˚
FEM	12.0	5.60	5.20	4.70	4.00
Das et al. (1976) Equation (1)	–	6.33	5.14	4.46	–
Chattopadhyay & Pise (1986) Equation (2)	–	9.83	9.92	8.74	–

confining stress, small-scale physical modelling under 1g conditions has little relevance to the behaviour of a full-scale prototype. However, if we reanalyze the 1g results based on the steady state line of the soil as described by Altaee & Fellenius (1994), the 1g model that prepared in a loose state will simulate a prototype model of dense state. So, if the 1-g model is prepared in a dense state, this will simulate a prototype of very hard soil which may not be exist in reality. If we considered this physical modelling view, the equation (1), which had been derived from a 1g loose sand model of φ = 31˚, will simulate the behaviour of a pile in dense sand. However, the equation (2), which had been derived from a 1g dense sand model of φ = 41˚, will simulate the behaviour of a pile in a stiffer soil than that can be found in field.

5 CONCLUSIONS

Steel pipe piles, embedded in saturated sand, have been subjected to static mooring forces using the finite element method. Based on the results in this paper, the following conclusions are made:

1. The ultimate resistance of a pile under oblique pull is a continuous function of the inclination of the pull and depends also on the net uplift and the ultimate lateral capacity of the pile.
2. Considering soil-pile interaction behaviour of piles plays a main role in defining the critical inclination angle of the load at which the failure changes from axial failure to lateral failure.
3. Comparing the present results with the previous theoretical models shows that most of the available models did not consider the prototype scale. So, they should be modified to be practically useful.
4. More scaled experimental work should be done to get the prototype scale behaviour. Using these experimental results some numerical parameters can be well estimated and a good numerical model can be designed to simulate the behaviour of pipe piles under mooring forces.

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