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RESEARCH ARTICLE

Effect of seabed slope on pile behaviour of fixed offshore platform under lateral forces

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Abstract Fixed offshore platforms supported by pile foundations are required to resist lateral load due to wave and current forces. The response to environmental loads is strongly affected by the soil structure (pile) interaction. The forces exerted by waves are most dominant among the lateral environmental forces which governing the jacket structures design especially the foundation piles. The present investigation is to perform a static wave analysis on a typical fixed offshore platform for extreme environmental conditions, and to study the effects of the combined lateral and vertical loads on pile group foundation. The three dimensional modeling and analysis of the offshore platform are done using finite difference method. The present analysis was done under static condition considering the structural and the environmental loads at extreme environmental conditions, by reaching the state of static equilibrium. A parametric study has been done by varying the seabed slope to examine the variation in soil-structure interaction behaviour of piles. It has been found that the lateral displacement at the pile top and at the seabed level increases as the seabed slope increases. It is also noticed that the depth at which the maximum shear force and bending moment occurs from the pile top increases as the slope of the seabed increases.

Keywords Fixed offshore platform · Soil-structure interaction · Static wave load · Finite difference analysis (FDA) · Lateral displacement · Bending moment

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1 Introduction

Piles are also subjected to significant amount of lateral loads and overturning moments besides axial load. Lateral loads are in the order of 10–15 % of the vertical loads in the case of onshore structures, and in the case of coastal and offshore structures these lateral loads can exceed 30 % of the vertical loads (Narasimha Rao et al. 1998). Hence, the foundation piles have a significant effect on the response of fixed offshore structures due to lateral loading. Among all the environmental forces, wave forces and forces due to ocean currents associated with the waves contribute the most to the total lateral forces experienced by the offshore structures. Therefore, proper attention has to be given in designing such pile structures of fixed offshore platforms under lateral loads.

Piles are commonly selected as a cost effective option for the support of raised structures, highway infrastructures, and offshore platforms. These structural members are often subjected to considerable lateral forces such as wind loads in hurricane prone areas, earthquake loads in areas of seismic activity, and wave loads in offshore environments. Soil-structure interaction is the mechanism that governs the pile response behaviour and the ultimate capacity of the structure for the applied loads. A common approach to the analysis of laterally loaded piles is the load-transfer approach, which involves treating the soil as a series of springs down the length of the pile. Upon defining the deformational characteristics of the soil medium, the flexural behaviour of the foundation and the condition at the interface, the soil-foundation interaction problem is basically reduced to the determination of the contact stress distribution at the interface. Once the contact stresses are determined, it is possible to evaluate the deflections, flexural moments and shear forces in the foundation and the corresponding stresses and displacements in the soil medium.

An oil platform or oil rig is a large offshore structure used to house, workers and machinery needed to drill and/or produce oil and natural gas through wells in the ocean bed. Exploration drilling in shallow water depths is mostly done with the help of a fixed jacket platform while floating structures are used for depths going as deep as 2,000 m. Based on the geometry and behavior, the offshore structures have been classified into fixed platform, compliant platform and floating production systems.

In the study of response of fixed offshore structures, the pile foundations have got much importance. The response to environmental loads is strongly affected by the soil—pile interaction. Among all the environmental forces, wave forces and forces due to ocean currents associated with the waves which contribute the most to the total lateral forces experienced by the offshore structures. And vertical loads due to crew quarters, drilling rigs, and production facilities etc. will be coming over the deck.

Sloped seabed surfaces are common at the seabed level in offshore due to the irregularities of the seabed which may form due to the action of water. Even after the installation of an offshore structure, there are chances for formation of slope along the seabed due to erosion, or due to the action of water. Therefore, it is important that the structure should be safe enough even in sloped seabed conditions. It is, therefore, essential to study the variation in soil-structure interaction for sloping seabed cases and plane seabed cases of offshore structures.

The deformation and stresses of an offshore concrete pile under combined structural and wave loading was analyzed by Eicher et al. (2003) using finite element method. A control model is analyzed using a specific set of control data, then compared to succeeding models as the pile and loading parameters are changed. The loading parameters used to complete this study are the wave period (T), the wave height (H), and the incident wave angle (α). The pile strength parameters used to complete the study are the mean concrete strength and the amount of steel reinforcement. The effect of different parameters on the response of a fixed offshore platform subjected to transient loading due to extreme wave and current loading was studied by Mostafa and Naggar (2004). The soil resistance to the pile movement is modelled using dynamic p–y (p-lateral load Vs y-lateral deflection) curves and t–z (axial load Vs axial settlement) curves to account for soil nonlinearity and energy dissipation through radiation damping. p- and z-multipliers are used to account for the pile–soil–pile interaction in a simplified way. Chae et al. (2004) has conducted several numerical studies performed with a three-dimensional finite element method (FEM). Model tests and a prototype test of a laterally loaded short rigid piles and pier foundation located near slopes were studied and the model test results were compared with the measurements of field tests. It was found that, due to the slope effect, the lateral resistance of the pile decreases, as the location of the pile is closer to the crest of the slope. The effect of seabed instability on a fixed offshore structure was examined by Mostafa and Naggar (2006). In his study, the seabed instability manifested in movement of soil layers, exerts lateral forces that may cause large stresses in offshore foundations. Many researchers have also studied the influence of ground slope on laterally loaded pile behaviour (Muthukkumaran et al. (2008); Almas Begum and Muthukkumaran (2009); Muthukkumaran (2014)).

From the literature review, it is clear that the study of response of fixed offshore platforms due to different types of load has received a good deal of attention from many researchers and practicing engineers. However, little study has been found in the literature on the response of fixed jacket platform due to wave and current loading including soil-structure (pile) interaction and seabed slope effect. As fixed jacket platforms are the most common type of platform used, it is, thus, in industrial and commercial interest that these platforms have a long life without succumbing to the harsh environment in which they have to perform. Fixed jacket platforms are very accident prone. Accidents have occurred in the past in all the four stages; during transportation, installation, operation and removal. The probability of accidents can be reduced by proper design and analysis of the structure. To provide a more accurate and effective design for pile foundation systems of a fixed jacket platform under axial and lateral loads, a finite difference model using FLAC 3D is created and analyzed. The wave loads acting on the structure is calculated using the corresponding environmental data and the structural load is applied together on the structure. The static analysis is done for applying both vertical and lateral load together. A parametric study is also done by varying the seabed slope under similar loading conditions.

2 Platform description

The fixed jacket offshore platform considered in this study is the 'Kvitebjørn' platform and the platform details have taken from Mostafa and Naggar (2004). The water depth at the site is 190 m and the substructure is a piled steel jacket. The Kvitebjørn substructure has four legs supported by vertical steel piles grouped symmetrically around each corner leg. The upper part of the structure is connected to the lower part through a traditional grouted connection and extends to approximately 25 m above the mean sea level (MSL). The jacket's lower part is approximately 45 m high and is connected to the pile foundation. The structure is levelled using four levelling piles and is permanently fixed on sixteen piles driven to about 90 m penetration depth.

The weights of the upper and lower parts of the structure are approximately 73,000 kN and 45,000 kN respectively. The total weight of the foundation is 53,000 kN and the

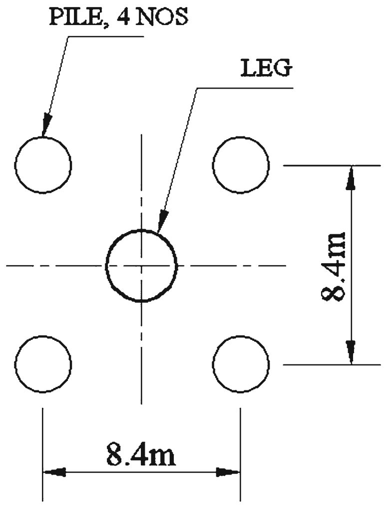


Fig. 1 Pile Group Layout for One Leg, after Mostafa and Naggar (2004)

total weight of the platform is 171,200 kN. The structure is designed to support a maximum operating topside weight of 225,000 kN. The lower part is square shaped with base dimensions 50 m × 50 m, and is approximately 45 m high and has vertical corner legs. The top part extends from approximately El. −145 to El. +8 m and has a constant batter on all sides with square dimensions at the bottom of 50 m × 50 m to square dimensions at the top of 25 m×25 m. The jacket is flared on two sides to meet the interface dimension of 22.5 m×30 m towards the topside at El. 21.2 m. These dimensions are held constant from El. 21.2 m to the topside interface elevation of 24.1 m. All elevations are relative to MSL. The jacket is supported on 16 piles with a diameter of 2.438 m arranged in symmetrical groups of four piles per corner leg. A typical pile group layout for one leg is shown in Fig. 1.

3 Environmental data

The environmental data are based on STATOIL specifications 'Metocean Design Criteria for Kvitebjørn'. The maximum directional wave heights for the 100-year return period are given in Table 1, including the mean wave period along with the 90% interval. The current associated with the 100-year return period design wave heights are given in Table 2. No associated wind has been specified. The thickness of marine growth is considered to be 20 mm below El. +2 m. The roughness due to marine growth is taken into consideration when determining the coefficients in Morison's equation (Morison et al. 1950) for wave forces. The average dry density of the marine growth material is considered to be 1300 kg/m^3. Drag and inertia coefficients are assumed to be 0.7 and 2.0 respectively, and the wave kinematics is calculated using the Stokes wave theory (Stokes 1847).

Table 2 Values for associated current, after Mostafa and Naggar (2004)

Depth below sea-level (m)	Current speed (cm/s)
0	50
25	50
50	50
75	46
100	42
125	39
150	36
175	32
190	29

4 Steps involved to find wave loads

Evaluation of the wave loads on a structure in the ocean involves the following:

(i) Identification of 'normal' and 'extreme' loading conditions
(ii) Choice of a suitable wave theory
(iii) Evaluation of water particle velocities and accelerations according to the chosen theory, which in conjunction with Morison's equation (Morison et al. 1950) lead to the quantitative evaluation of the force.

4.1 Environmental considerations

Normal environmental conditions (conditions that are expected to occur frequently during the life of the structure) are important both during the construction and the service life

Table 1 Design waves versus return period, after Mostafa and Naggar (2004)

Return period (year)	Wave height (H) in meter	Height above MSL (a) in meter	Wave period	
			Mean value (T)	90 % interval
1	22.5	12.8	13.8	12.2–15.5
10	25.3	14.2	14.6	13.0–16.4
100 (extreme condition)	28.5	16.1	15.3	13.6–17.1
10,000	36	20.4	17.1	15.1–19.1

Table 3 Soil properties formulated for *FLAC 3D* input after (Mostafa and Naggar 2004, 2006)

Soil layer data			*FLAC 3D* input					
Sl. no.	Depth (m)	Type of soil	E (kPa)	G (kPa)	K (kPa)	φ (degrees)	c (kPa)	Shear strength (kPa)
1	7.5	Very soft to soft silty sandy clay	3,000	1,111.11	3,333	0	15	15
2	24.5	Sandy, clayey silt	40,000	14,814.8	44,444	0	80	80
3	15	Very stiff to hard silty clay	180,000	66,666.7	200,000	0	150	150
4	5	Very dense fine sand	100,000	40,000	66,666	35	–	300
5	10	Very stiff to hard clay	435,000	161,111	483,333	0	290	290
6	15	Very stiff to hard clay	277,500	102,778	308,333	0	185	185
7	8	Very stiff to hard clay	292,500	108,333	325,000	0	195	195

of a platform. The extreme environmental conditions (conditions that occur quite rarely during the life of the structure) are important in formulating platform design loads. For extreme condition of waves, definition of the extreme sea-states should provide an insight as to the number, height, and crest elevations of all waves above a certain height that might approach the platform site from any direction during the entire life of the structure. Here, in this study, the extreme environmental conditions are chosen for the analysis. i.e., the environmental data for return period of 100 year is considered. Accordingly, the extreme environment wave length was estimated. From the environmental data for extreme environmental conditions, and using the limits of validity for selected wave theory (Le Mehaute 1969), it is found out that the region comes in the area of deep water waves. Therefore, Stokes wave theory (Stokes 1847) is used for the calculation of water particle velocity and acceleration (wave kinematics).

Morison's equation (Morison et al. 1950) together with API (2005) wave force guidelines is used to generate the wave forces. Since the currents are associated with the waves, due consideration given to the possible superposition of current and waves. So the current velocity added vectorally to the wave particle velocity before the total force is computed using Morison's equation (Morison et al. 1950). Using the above procedure, the total lateral loads due to waves and current associated with the waves are calculated at different heights of the platform from the seabed level to the mean sea level.

5 Modelling of fixed jacket offshore platform

Numerical models involving FDA can offer several approximations to predict true solutions. Often the problem being modelled is complex and has to be simplified to obtain a solution. Here finite difference analysis (FLAC 3D) is used for the analysis. An explicit time-marching finite difference solution scheme is used for the analysis and the calculation sequence is, 1. Nodal forces are calculated from stresses, applied loads and body forces, 2. The equations of motion are invoked to derive new nodal velocities and displacements, 3. Element strain rates are derived from nodal velocities and 4. New stresses are derived from strain rates, using the material constitutive law. This sequence is repeated at every time step, and the maximum out-of-balance force in the model is monitored. This force will either approach zero, indicating that the system is reaching an equilibrium state, or it will approach a constant, nonzero value, indicating that a portion (or all) of the system is at steady-state (plastic) flow of material.

Multilayered soil block is modelled using brick mesh shape elements which has 8 reference points or nodes. Each structural element entity is composed of three components: nodes; individual elements (called SELs); and node/grid links. The characteristics of each of these components distinguish the behavior of the beam, pile and shell entities. The soil layers are modeled with Mohr-Coulomb plastic model. Since the analysis was carried out using Mohr-Coulomb soil model, the consolidation behaviour of soil on pile response is not studied in this paper. The soil profile considered is a multilayered soil block. The Poisson's ratio is taken as 0.25 for sand and 0.35 for clay. From the undrained cohesion (c_u) and angle of internal friction (ϕ) values (Mostafa and Naggar 2004, 2006), Poisson's ratio and type of soil layers, the properties which has to be given as inputs for FLAC 3D are formulated and presented in Table 3. These properties include Modulus of Elasticity (E), Shear Modulus (G), Bulk Modulus (K), Angle of internal friction (ϕ), cohesion (c) and shear strength.

The piles in each group are fixed to a rigid cap which is modelled as pile cap plate using shell elements, and the deck plate modelled using shells which is represented by three-noded flat triangular elements. Each beam or pile element has two nodes and each node has 6 degrees of freedom (3 rotations and 3 translations at each node). Each element has 12 active degrees of freedom in total. Each shell element has 3 nodes and each node has 6 degrees of freedom which leads to a total of 18 degrees of freedom (3 rotations and 3 translations at each node) in the shall element. The stiffness matrix of the beam or pile element includes all six degrees of freedom at each node to represent axial, shear and bending action within a beam structure.

The soil nodes and pile nodes are connected by bilinear Mohr-Coulomb interface elements. This allows an approximate representation of the development of lateral resistance with relative soil-pile movement and ultimately the full limiting soil pressure acting on the piles. Piles and the platform legs are assigned as pile and beam members respectively which are modelled as two nodded structural element segments (Material–Steel; Young's Modulus, E 2×10^{11} N/m^2; Poisson's ratio,ν 0.3; Diameter of pile, d 2.438 m; Pile length 100 m; Embedded pile length 90 m). Horizontal members and bracings are also modelled as beam members. Two nodded, linear elements represent the behaviour of beams and piles. Piles interact with the grid via shear and normal coupling springs. The coupling springs are nonlinear, spring-slider connectors that transfer forces and motion between the pile and the grid at the pile nodes (by way of the link emanating from each pile node). The spring constants are calculated using Vesic's equation (Vesic 1961).

The 3D view of the model generated is shown in Fig. 2. The top of the model, at $z = 0$, is a free surface. The base of the model, at $z = 500$ m, is fixed in the z-direction, and roller boundaries are imposed on the sides of the model, at x = 150 m and $y = 150$ m. In order to simulate the sloping ground in the model, the slope angle has been introduced in the x direction which is against the lateral load. The model is first brought to an equilibrium stress-state under gravitational loading before the installation of the pile. In case of sloping ground, the slope has been introduced in the model before installation of piles. In the next stage of analysis, the model is brought into equilibrium after the installation of the pile.

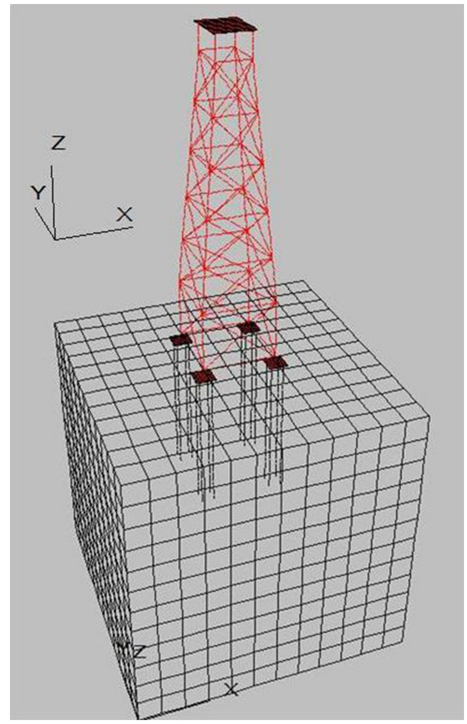


Fig. 2 3D view of the finite difference model generated

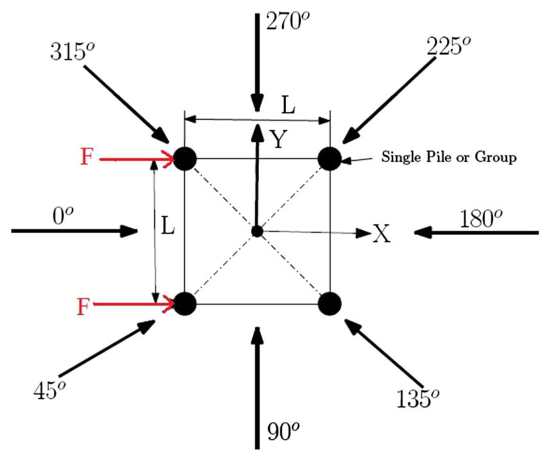


Fig. 3 Application of wave load (F) in plan

6 Application of loads

The structural load and the total wave load are calculated for the extreme environmental conditions which have been used for the static analysis. The total structural load intensity is estimated as 2,30,000 kN (Mostafa and Naggar 2004). This load is applied as a vertical pressure on the top of the platform (deck plate) as shown in Fig. 4. The total lateral load (F) caused by waves and associated current was estimated as per the above theory and it is assumed that the wave load acts in the x-direction which is also against the slope seabed. The calculated forces have been applied as equivalent static force (the hydrodynamic effect has not been considered in the present analysis) as shown in Figs. 3 and 4. The maximum force will be at the MSL and than reduces with depth of water. Since the results are very much comparable with Mostafa and Naggar (2004) results even without considering the hydrodynamic aspect and hence the equivalent static analysis is sufficient in order to reduce the computational time.

7 Static solution by finite difference analysis

A certain number of steps are required to arrive an equilibrium (or steady-flow) state for a static solution. A static or steady-state solution is reached in the finite differ-

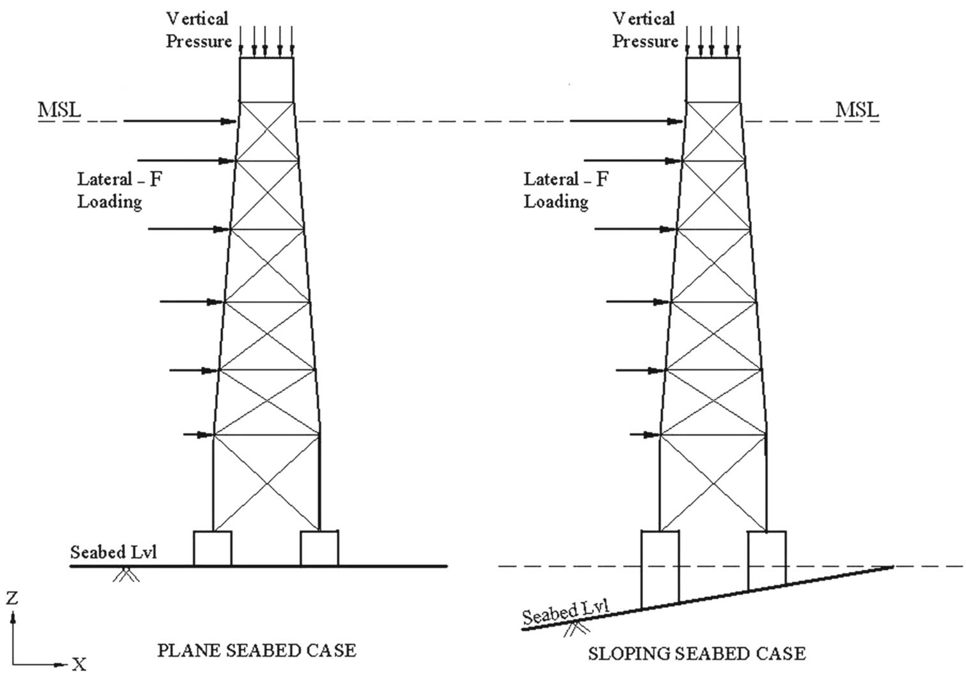


Fig. 4 Application of Loads in Elevation

ence analysis when the rate of change of kinetic energy in a model approaches a negligible value. This is accomplished by damping the equations of motion. At the conclusion of the static solution stage, the model will be either at a state of equilibrium or at a state of steady flow of material, if a portion (or all) of the model is unstable (i.e., fails) under the applied loading conditions. A model is in exact equilibrium if the net nodal-force vector (the resultant force) at each grid point is zero. In the static analysis of fixed offshore platform, the unbalanced force history reduces to a much lesser value, and the displacement history at any node becomes constant which indicates that the state of static equilibrium has been reached.

8 Behaviour of piles on loading for plane seabed case

The static analysis of the fixed offshore platform has been done for the plane seabed case for the loadings mentioned above, and the following results were obtained. The legend followed for the platform legs and piles in the group are shown in Fig. 5. For the response study, the piles L1P2, L1P3, L2P2 and L2P3 are chosen, which are the front and rear piles of Leg1 (which is in tension) and Leg2 (which is in compression). According to the direction of loading (x-direction as shown in Fig. 4.), the front and rear piles of legs are mentioned here. These four piles in a row in x-direction are chosen for response study, since the other piles will also act symmetrically due to the same direction of lateral loading.

The four piles considered for the response study are,

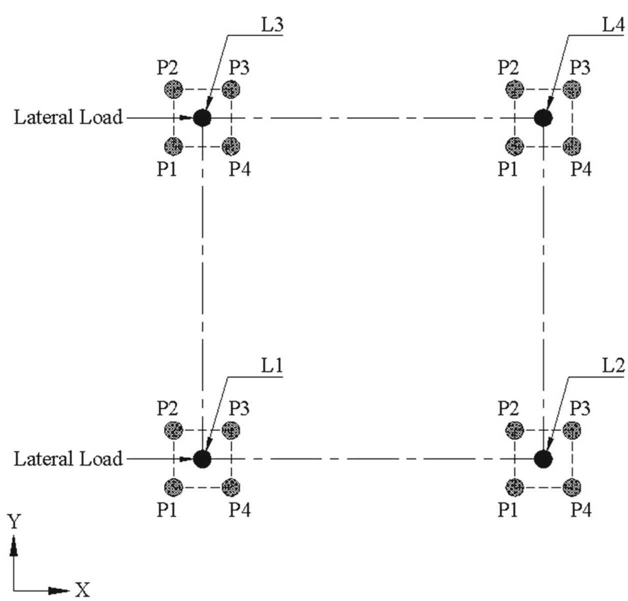


Fig. 5 Legend followed for platform legs and piles

L1P2 -Front pile of Leg1
L1P3 -Rear pile of Leg1
L2P2 -Front pile of Leg2
L2P3 -Rear pile of Leg2

8.1 Validation of results

The present FDA study results were compared with the results of a similar study done by Mostafa and Naggar (2004). The lateral displacement and bending moment for a pile obtained are compared and shown in Figs. 6 and 7. From

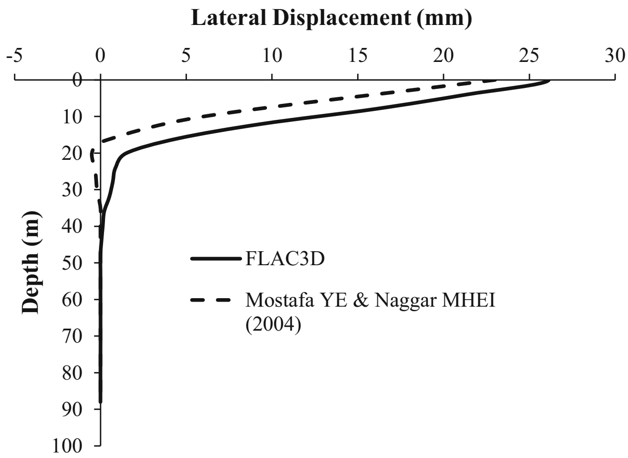


Fig. 6 Lateral displacement comparison

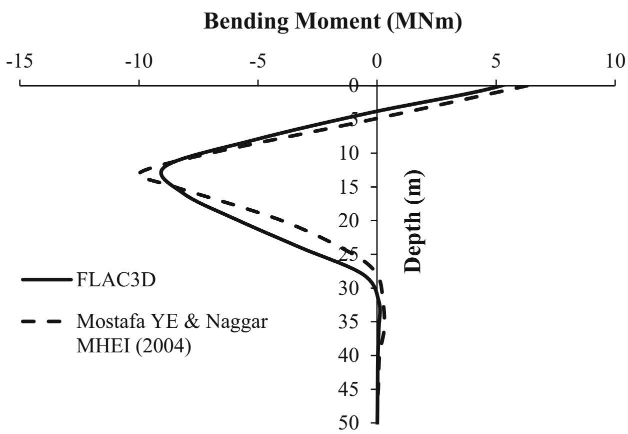


Fig. 7 Bending moment comparison

the comparison, it is found that the value of maximum displacement of pile at seabed surface and bending moment of the present study has very good agreement with Mostafa and Naggar (2004) results. The mesh convergence (mesh sensitivity) analysis was also carried out for different mesh sizes including mesh refinement near the piles. However, there is no significant change in the result and hence the present mesh is used for further analysis with minimum computational time. And also the mesh which gives comparable result with Mostafa and Naggar (2004) was finalized for further analysis.

8.2 Comparison of response of piles in the group for plane seabed

In order to understand the behaviour of legs and piles within the leg in plane seabed, the lateral displacement of pile 2 & 3 of legs 1 & 2 were compared which is shown in Fig. 8. The zero value in the y-axis of these graphs shows the seabed level and the pile extends to 12m above the seabed to the pile cap. From these graphs, the maximum values for lateral displacement and bending moment can be compared for these piles in a row, in the direction of lateral load. The maximum lateral displacement at seabed level among all the piles has occurred in the rear piles of leg1 and leg2 (i.e. L1P3 & L2P3). The maximum lateral displacement of L2P3 is 48mm whereas L1P3 is 35mm. However, the front pile displacements of both the legs (legs 1 & 2) are same of 28mm under a plane seabed condition. This is due to the reason that, the moment developed due to the applied lateral load will be more at the top of rear piles than that of front piles. And also it will be expected that, the vertical load coming over the rear piles will be more than front piles due to the lateral displacement of the piles. Therefore, the rear piles show more lateral displacement than front piles in both the legs.

Figure 9 shows the bending moment variation of piles in the legs. It is clearly seen that the maximum bending moment has occurred in the same pile of L2P3 as like in the lateral displacement. The influence of lateral load in the platform performance is significantly affected by the soil-pile interaction which is very clearly seen in the figure. It is also observed that the occurrence of (depth of fixity) maximum bending moment below seabed significantly changes with pile to pile in a leg or same piles in different legs. The depth of fixity of rear pile in leg 1 and 2 is about 5.7D and 6.4D (D is diameter of pile) respectively where these values in front piles of both the legs are almost same of 3.7D.

The rear piles bending moment are much higher than front piles and also the depth of fixity of rear pile are nearly two times more than front pile depth of fixity. This shows very clearly that, the offshore platforms under these kinds of lateral load are very critical in the stability and hence the piles and the structure need to be designed for the critical state even though the pile sizes are same.

9 Parametric study by introducing seabed slope

The slope is introduced downwards opposite to the direction of lateral loading and the downward slope is assumed to start from the top rear end point of the soil block with reference to the loading direction (i.e., along $x-$direction). The introduction of seabed slope is shown in Fig. 10. Generally, offshore seabed has mild slope to steeper slopes with respect to the depth of water and water current forces. The seabed slope also depends on the nature of seabed soils. In order to consider the seabed slope effect in the platform behaviour, the static analysis of the platform is done for different seabed slopes of 1 in 50, 1 in 25, 1 in 10 and 1 in 5 under the same loadings. The analysis results were compared with plane seabed slope which clearly brought out the importance of seabed slope in the offshore platform behaviour. Since the geostatic stress and passive resistance in front of the pile is reduced in sloped seabed case, there is significant increase

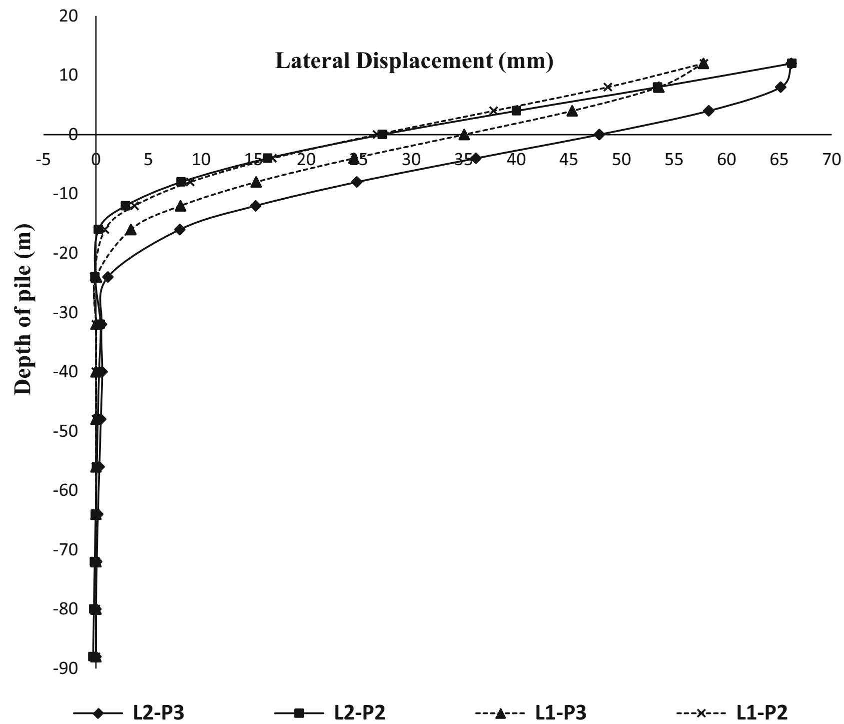


Fig. 8 Lateral displacement vs. depth of pile for plane seabed

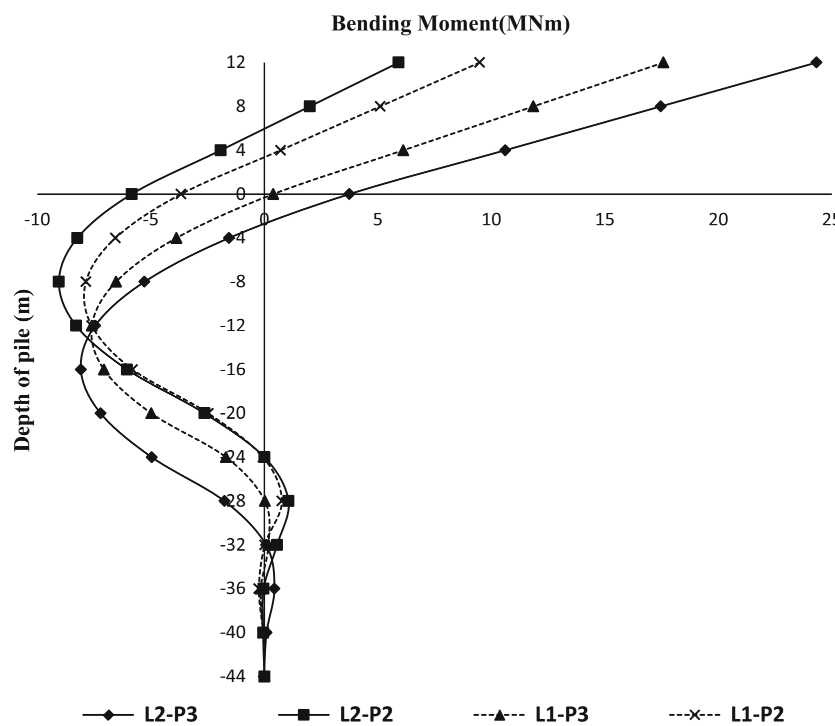


Fig. 9 Bending moment vs. depth of pile

in pile lateral deflection and bending moment. In the present study, the effect of scour is considered in the pile response since the sloped seabed itself subjected to lateral slinging. Therefore, the scour influence may not be a significant role in the pile response on sloped seabed.

9.1 Effect of seabed slope on lateral displacement

The lateral displacement versus depth of pile for the piles L2P3, L2P2, L1P3 and L1P2 are shown in Figs. 11, 12, 13 and 14 respectively. From these figures, it is very clear that,

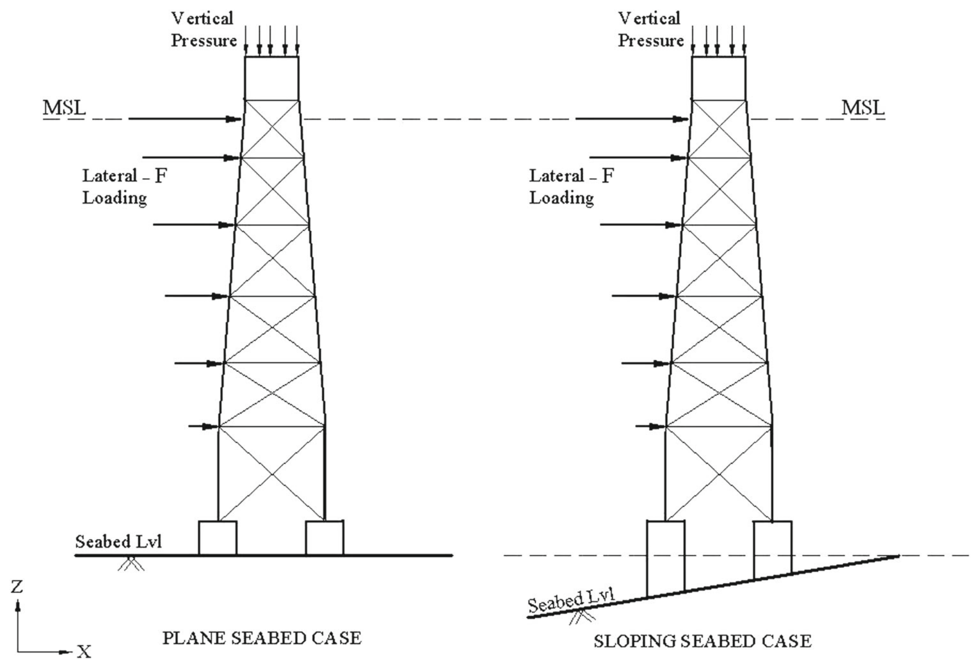


Fig. 10 Diagram showing direction of slope introduced in seabed

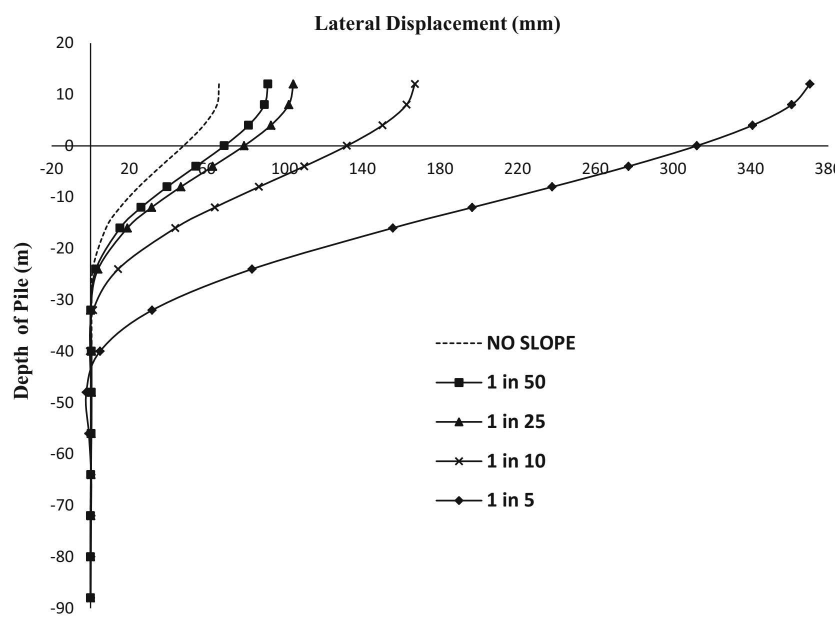


Fig. 11 Lateral displacement vs. Depth of pile (L2P3)

the rear pile of leg2 is showing the maximum lateral displacement in all sloped seabed conditions. From these figures, it is very clear that the lateral displacement of piles are not much significant up to a seabed slope of 1 in 10. However, the increase in seabed slope beyond 1 in 10 has very significant impact in the lateral response of the piles irrespective of the location (rear or front). It is also noticed that, the influence of slope not only increases the lateral displacement, it also disturbs the overburden soil (top soil) deeper than the plane seabed condition. This is very important observation as per axial capacity is concerned. Since the axial capacity estimated is based on side frictional resistance also, there will be a significant reduction in the side friction. For instance, the lateral displacement is almost negligible depth at 28m below seabed level in plane seabed whereas this is nearly 42 m in 1 in 50 seabed slope (Fig. 11).

Figure 15 shows the lateral displacement comparison with different seabed slopes. It has been observed that, the lateral displacement of piles at seabed level increases with increase in seabed slope. This is due to the reason that, the increase in seabed slope decreases the passive resistance mobilized in front of the pile and also the unstable slope lead to slide the top soil laterally towards piles. Based on the analysis, it is very clear that the L2P3 (leg 2, pile 3) pile is the critical

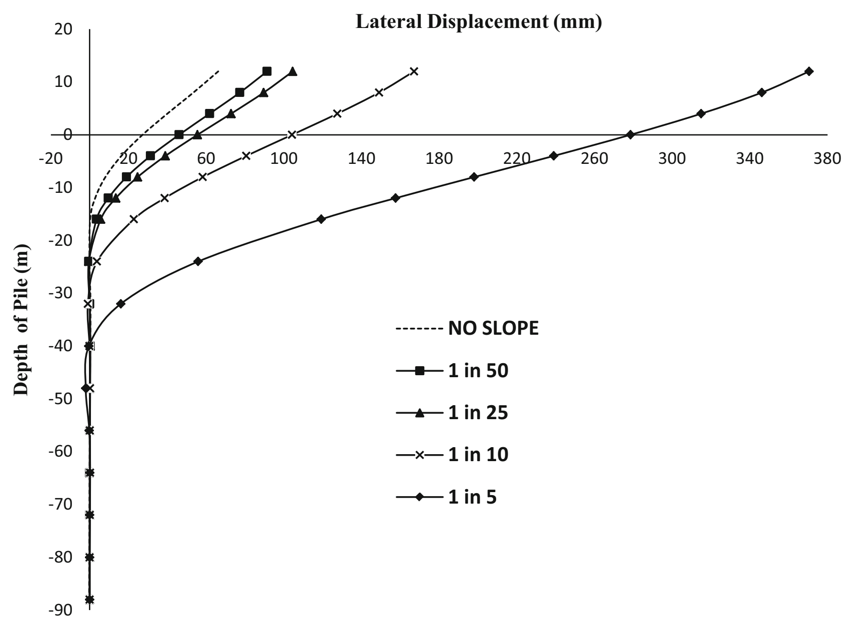


Fig. 12 Lateral displacement vs. depth of pile (L2P2)

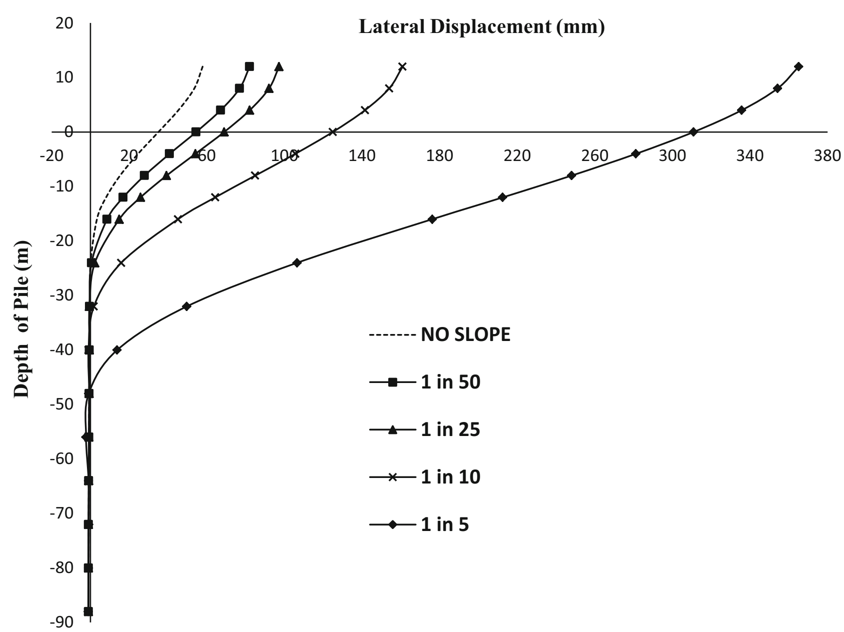


Fig. 13 Lateral displacement vs. depth of pile (L1P3)

one compared to all other piles. If we see the behaviour of L2P3 pile under various slope seabed conditions, the lateral displacement is nearly 6 times more in 1 in 5 slope seabed than plane seabed. However, the seabed slope up to 1 in 25, the magnitude of lateral displacement is not much significant. From this observation it may be concluded that if the seabed slope is flatter than 1 in 25, the slope effect may be neglected and there may not be any significant change in the offshore pile design. However, if the seabed slope is more than 1 in 25 (steepness), the effect of seabed slope needs to be considered in the offshore pile design which is very clearly shown in the lateral displacement diagram (Fig. 15). The sliding soil mass (in case of steeper seabed slope) will generate high inertial and drag forces on the piles which will significantly reduces the lateral as well as the axial capacity of the piles. In the general pile design this particular aspect may not be considered always and this may be a critical aspects as per as the offshore pile design in sloped seabed concern.

9.2 Effect of seabed slope on bending moment

In general, the piles were designed for the maximum bending moments. Normally the fixed head piles will have two maximum moments one at the pile top (i.e. pile and pile cap

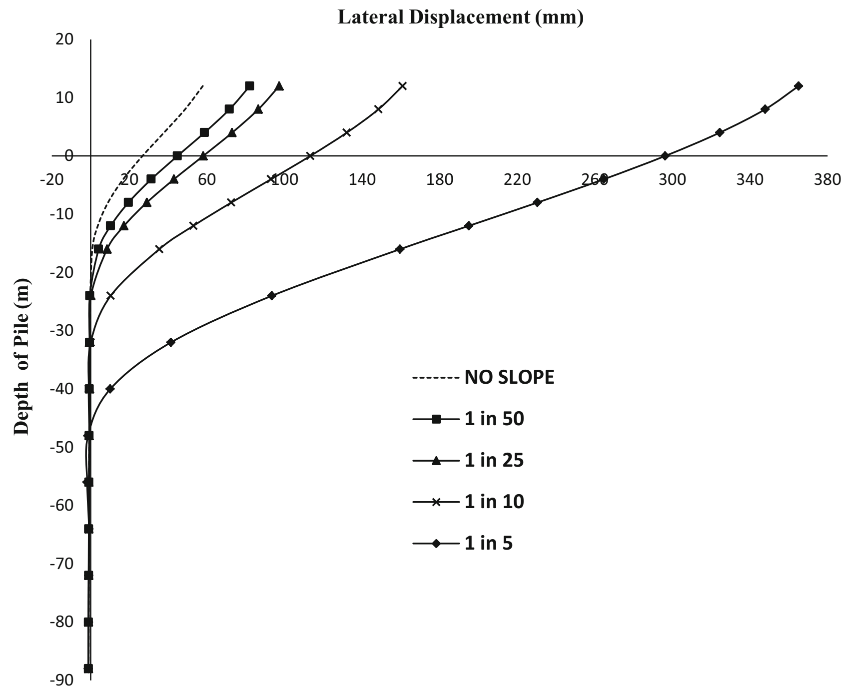


Fig. 14 Lateral displacement vs. depth of pile (L1P2)

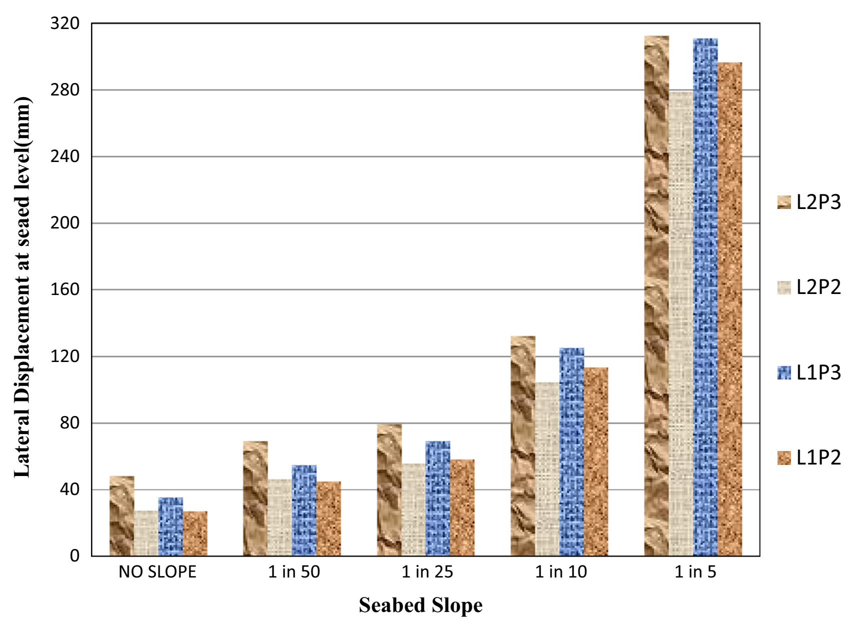


Fig. 15 Effect of seabed slope on lateral displacement of piles

connection) and the other one some depth below the seabed or ground which is normally called as depth of fixity. In general, the bending stress developed in the pile should be well within the allowable bending stress of the pile materials and this plays an important role in the pile structural stability. In order to understand the bending behaviour of piles, the bending moment results are plotted against the depth of piles for the piles L2P3, L2P2, L1P3 and L1P2 are shown in Figs. 16, 17, 18 and 19 respectively. From these figures, it is very clearly seen that the seabed slope has very significant effect in the pile moment carrying capacity or moment of resistance which is the direct measure of pile structural stability.

The seabed slope has very high influence on the pile head bending moment (moment developed between pile and pile cap joint) rather than moments at the fixity depth. Since the passive resistance reduction in sloped seabed has significant influence in the pile head moment in addition to additional lateral load due to lateral soil movement in steeper slopes. The seabed slope not only affected the moment carrying capacity of the piles it also significantly influenced the depth of fixity.

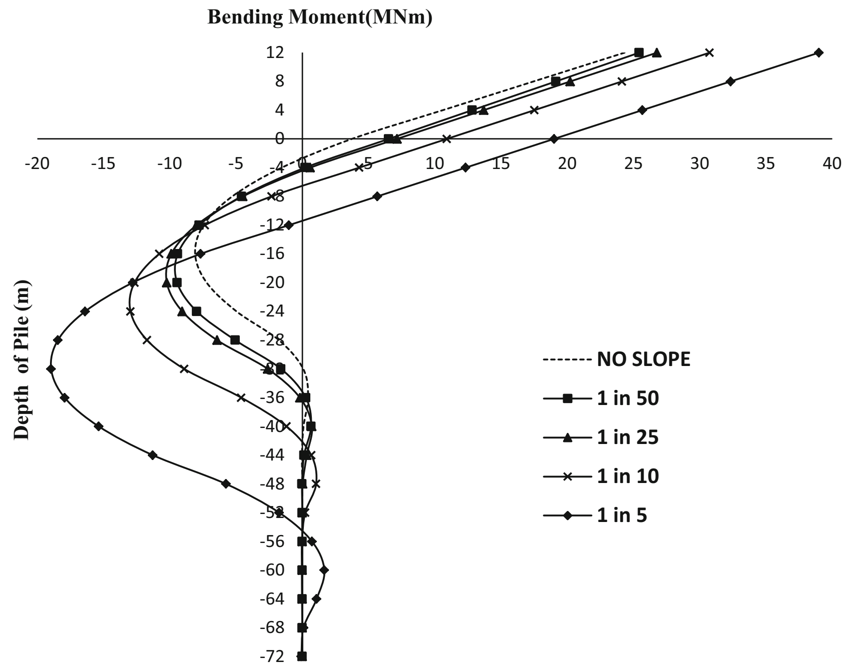


Fig. 16 Bending moment vs. depth of pile (L2P3)

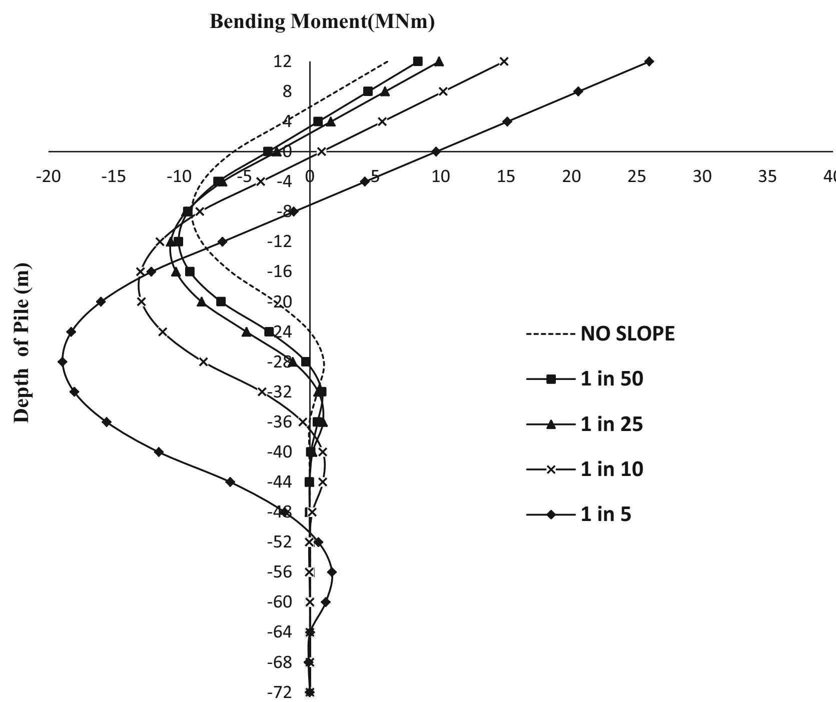


Fig. 17 Bending moment vs. depth of pile (L2P2)

As we know, in the conventional pile design, the piles will be designed as a cantilever beam by knowing the depth of fixity. If the seabed changes from plane to 1 in 5 slope, the depth of fixity is increased by twice in case of L2P3 pile (Fig.16). Similar kind of behaviour was observed almost in all the piles. This shows that the developed moment may be very close to the moment of resistance of the pile or some time may be more than that and finally which leads to crack the pile members and the stability of the offshore platform.

Figure 20 shows the maximum bending moment comparison with different seabed slopes. It has been observed that, the developed maximum bending moment increases with increase in seabed slope. The maximum bending moment variation is not significantly affected by the seabed slope in case of flatter slopes up to 1 in 10 and beyond this seabed slope, the magnitude of bending moment is significantly affected by seabed slope. It is also observed that, the influence of steeper slope has neglected the vari-

Fig. 18 Bending moment vs. depth of pile (L1P3)

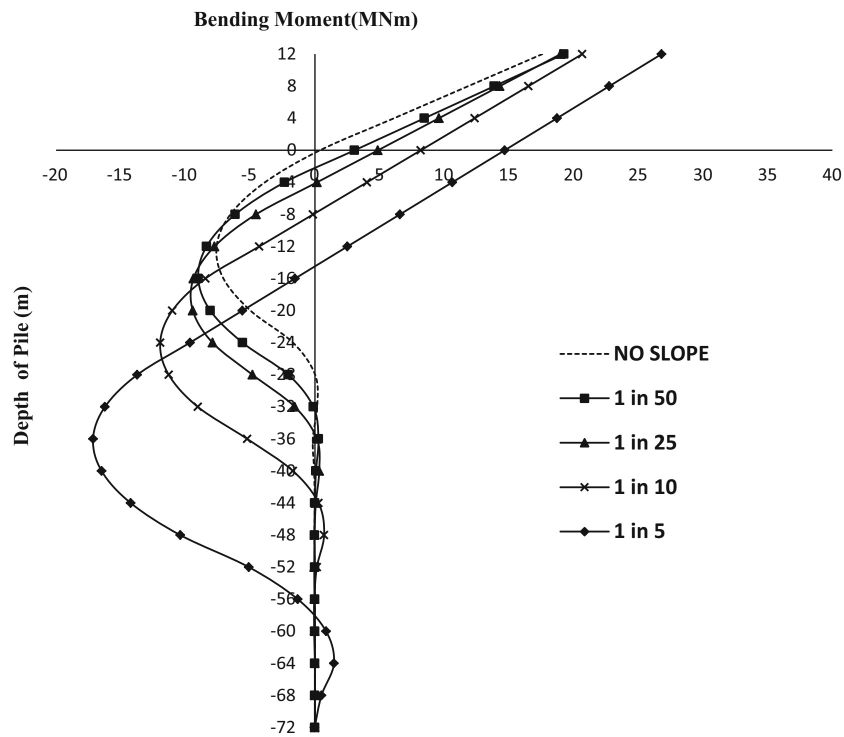
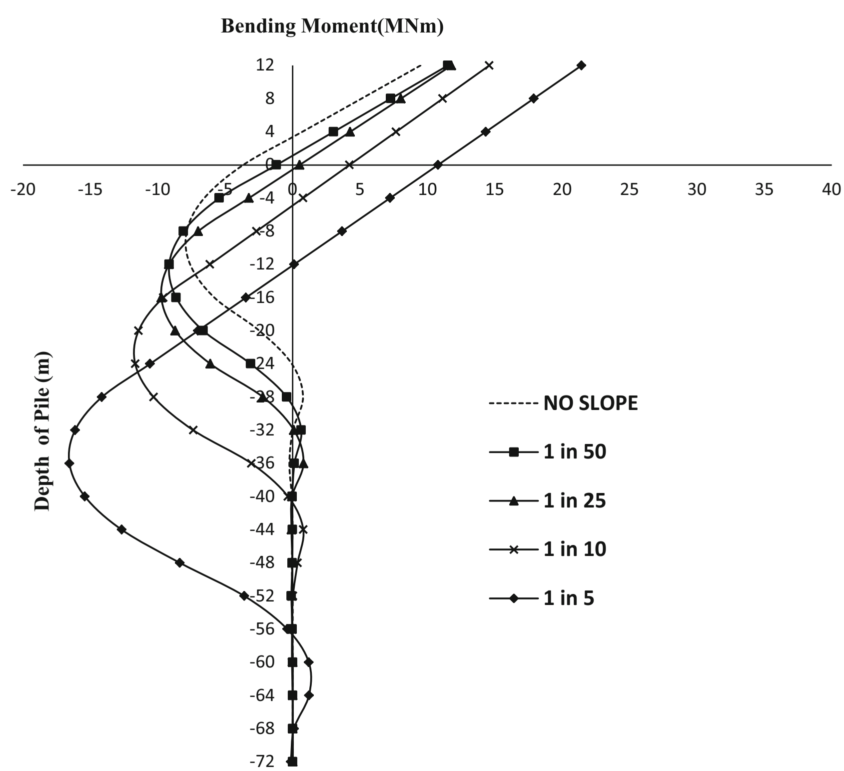


Fig. 19 Bending moment vs. depth of pile (L1P2)



ation in rear and front pile behaviour. The magnitude of L2P3 and L2P2 are almost same whereas this has very slight variation in leg 1 for 1 in 5 steeper seabed slopes.

10 Conclusions

A three dimensional finite difference model of the fixed jacket offshore platform has been modeled and satisfactorily vali-

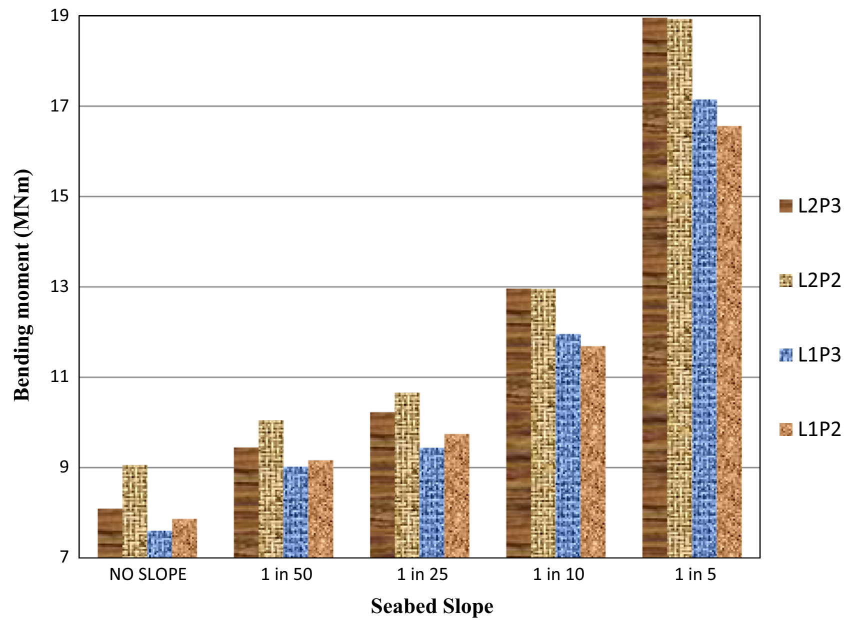


Fig. 20 Effect of seabed slope on maximum bending moment of piles

dated with Mostafa and Naggar (2004). From the analysis, it is very clear that the seabed slope has significant impact on the offshore pile design in terms of the lateral displacement and bending moment. The following important conclusions were drawn based on the above study,

- From the analysis, it is concluded that, if the seabed slope is flatter than 1 in 20, the seabed slope may not have significant influence in the pile behaviour and hence the flatter slope seabed may treated as plane seabed conditions and the pile design may be done accordingly.
- The steeper slopes (slope steeper than 1 in 10) not only influenced the lateral pile behaviour but also influenced (reduced) the axial capacity since the top soil disturbance increases in steeper slope up to 40 to 45m depth whereas this is only 25 to 30m depth in plane seabed condition.
- In steeper seabed slopes (1 in 5 slope), the piles may experience almost twice the bending moment of plane seabed conditions. This aspects needs to be considered in the offshore pile design in steeper seabed slopes.
- Steeper seabed slopes significantly influenced on the depth of fixity of the piles. The depth of fixity is almost twice the depth of fixity of plane seabed in 1 in 5 steeper slopes. Consequently, the magnitude of maximum bending moment is also twice.
- From the analysis it is clear that the stepper slope (1 in 5 slopes) has neglected the variation in rear and front pile behaviour. The magnitude of L2P3 and L2P2 are almost same in steeper slope whereas this has significant variation in plane seabed conditions. Therefore, it is concluded that this particular aspects need to be considered in the offshore pile design.

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References

Almas Begum N, Muthukkumaran K (2009) Experimental investigation on single model pile in sloping ground under lateral load. Int J Geotech Eng 3(1):133–146

Chae KS, Ugai K, Wakai A (2004) Lateral resistance of short single piles and pile groups located near slopes. Int J Geomech ASCE 4(2):93–103

Eicher JA, Guan H, Jeng DS (2003) Stress and deformation of offshore piles under structural and wave loading. J Ocean Eng 30:369–385

Le Mehaute B (1969) An introduction to hydrodynamics and water waves: water wave theories, Essa Technical report ERL 118-POL-3-2, vol. 2, U.S. Dept. of Commerce, Environmental science services administration, Pacific Oceanographic Laboratories, Miami, FL

Morison JR, O'Brien MP, Johnson JW, Schaaf SA (1950) The force exerted by surface waves on piles. Petrol Trans AIME 189:149–154

Mostafa YE, El Naggar MH (2004) Response of fixed offshore platforms to wave and current loading including soil–structure interaction. Soil Dyn Earthquake Eng 24:357–368

Mostafa YE, El Naggar MH (2006) Effect of seabed instability on fixed offshore platforms. Soil Dyn Earthquake Eng 26:1127–1142

Muthukkumaran K, Sundaravadivelu R, Gandhi SR (2008) Effect of slope on P–Y curves due to surcharge load. Soils Found 48(3):361–365

Muthukkumaran K (2014) Effect of slope and loading direction on laterally loaded piles in cohesionless soil. Int J Geomech ASCE 14(1):1–7

Narasimha Rao S, Ramakrishna VGST (1998) Influence of rigidity on laterally loaded pile groups in marine clay. J Geotech Geoenviron Eng ACSE 124(6):542–549

Recommended practice for planning, designing and constructing fixed offshore platforms—working stress design (2005) American Petroleum Institute Recommended practice 2A-WSD, 21nd edn. Washington, DC

Stokes GG (1847) On the theory of oscillatory waves, Transactions 8. Cambridge Philosophical Society, Cambridge

Vesic AS (1961) Beams on elastic subgrade and the Winkler's hypothesis. 5th ICSMFE 1:845–850