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Pile-soil-structure interaction effect on structural response of piled jacket-supported offshore platform through in-place analysis

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Abstract. In-place analysis for offshore platforms is essentially required to make proper design for new structures and true assessment for existing structures, in addition to the structural integrity of platforms components under the maximum and minimum operating loads when subjected to the environmental conditions. In-place analysis have been executed to check that the structural member with all appurtenance's robustness have the capability to support the applied loads in either storm or operating conditions. A nonlinear finite element analysis is adopted for the platform structure above the seabed and pile-soil interaction to estimate the in-place behavior of a typical fixed offshore platform. The SACS software is utilized to calculate the dynamic characteristics of the platform model and the response of platform joints then the stresses at selected members, as well as their nodal displacements. The directions of environmental loads and water depth variations have significant effects in the results of the in-place analysis behavior. The most of bending moment responses of the piles are in the first fourth of pile penetration depth from pile head level. The axial deformations of piles in all load combinations cases of all piles are inversely proportional with penetration depth. The largest values of axial soil reaction are shown at the pile tips levels (the maximum penetration level). The most of lateral soil reactions resultant are in the first third of pile penetration depth from pile head level and approximately vanished after that penetration. The influence of the soil-structure interaction on the response of the jacket foundation predicts that the flexible foundation model is necessary to estimate the force responses demands of the offshore platform with a piled jacket-support structure well.

Keywords: offshore platform; SACS; storm condition; pile soil interaction; in-place analysis.

1. Introduction

The Gulf of Suez area is the oldest and major oil producing from offshore in Egypt and an important shipping route for oil and commercial products. It contains a lot of oil fields, with significant oil reserves present in the subsurface; these features make the Gulf of Suez an economically valuable region. The gulf is rather shallow with depth range up to 100 m. Improvements in the oil and gas recovery from several fields have raised the interest for using these platforms well beyond their intended design life. Life extension of an existing jacket platform needs proper reassessment of its structural members, such as piled foundations. offshore structures have the added complication of being placed in an ocean environment, where the hydrodynamic interaction effects and dynamic response become major considerations in their design (Gudmestad 2000, Haritos 2007). Assessment of jacket platforms subjected to environmental loads greater than their original design loading frequently indicates that the capacity of the structural system is governed by the foundation (Nour El-Din and Kim 2015). There were several platforms damaged in hurricanes, where foundation damages or failures have been reported as could be seen in Fig. 1 (Aggarwal *et al*. 1996, Bea *et al*. 1999, Abdel Raheem 2014, Ishwarya *et al*. 2016). A total of 337 failure modes have been identified and analyzed by experts representing approximately 70% of the European offshore market to assess potential benefits of condition monitoring systems (Scheu *et al*. 2019). Krieger *et al*. (1994) described the process of assessment of existing platforms. Petrauskas *et al*. (1994) illustrated the assessment of structural members and foundation of jacket platforms against metocean loads. Craig and Digre (1994) explained assessment criteria for various loading conditions. Ersdal (2005) evaluated the possible life extension of offshore installations and procedures of standards, with a focus on ultimate limit state analysis and fatigue analysis. Gebara *et al*. (2000) assessed the performance of the jacket platform under subsidence and perform ultimate strength and reliability analyses for four levels of sea floor subsidence. It is important to include the wave load to ensure that the structural integrity of the offshore platform meet the design and assessment requirements (Golafshani *et al*. 2009, Abdel Raheem *et al*. 2012, Elsayed *et al*. 2015, 2016).

Offshore structures should be designed for severe environmental loads and strict requirements should be set

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Fig. 1 Platform with suspected foundation failure (Aggarwal *et al*. 1996, Ishwarya *et al*. 2016)

Fig. 2 Map of the studied offshore platform location

for the optimum performance (Abdel Raheem and Abdel Aal 2013, Abdel Raheem 2016). Design calculations for offshore structures require a mathematical model which is based upon the state of the art in offshore technology. In order to limit the complexity to an appropriate level for the engineering application; an approach was developed emphasizing aspects that are most relevant to bottom-mounted offshore structures. The first premise in the design of jackets is that the jacket natural period is well separated from the wave periods normally encountered in the in-place condition (Sadian and Taheri 2016, 2017), this ensures that the structure responds in a statically and not dynamically to the imposed wave loading. In such a case the structure can be analyzed for the forces imposed on it quasi-statically. In case the structure natural frequency approaches the predominant wave frequency then the analysis must take care of response amplification at the wave period (Abdel Raheem 2013, Khandelwal 2018). Mostafa and El Naggar (2004) undertook a parametric study of the soil-structure interaction on the response of a jacket structure subjected to transient loading due to extreme waves and currents, it concluded that the response of a jacket offshore tower is affected by the flexibility and nonlinear behavior of the supporting piles, and the response to environmental loads is strongly affected by the pile–soil–pile interaction (Shi *et al*. 2015). Asgarian *et al*. (2012) investigated the effect of soil-pile structure interaction on dynamic characteristics of the offshore platform through a comparison of experimental and numerical dynamic responses of a prototype jacket offshore platform for both hinge-based and pile supported boundary conditions. It is observed that dynamic characteristics of the system changes significantly due to soil pile-structure interaction.

This paper aims to investigate the pile-soil-structure interaction effect on the response of fixed offshore structures with a jacket foundation using in-place analysis. A case study of the existing fixed offshore platform located in Gulf of Suez by in-place strength analysis is considered. The simulation of offshore platform model and parameters setting are studied to distinct the data required for analysis and design of the offshore platform. The model of offshore platform structure which includes the topside platform and the support structure is elaborated including aspects of structure modeling, piled structures, hydrodynamic loading and environmental parameters for the site location of platform under consideration. A nonlinear dynamic analysis is formulated for reliable evaluation of fixed Jacket platform responses under environmental loads. A three-dimensional finite element model is formulated to determine the stresses and displacements in the jacket-type pile foundation under combined structural and environmental loadings. The horizontal components of the wave velocity and acceleration fields are multiplied by a wave kinematics factor that is intended to account for spreading direction and irregularity of the wave profile. The wave and current forces acting on the member is computed using Morison's equation, which decomposes the total force into an inertia component that varying linearly with the water particle acceleration and a drag component that varying quadratically with the water particle velocity. The analysis considers various nonlinearities produced due to change in the nonlinear hydrodynamic drag force. Numerical results are presented for various combinations of typical sea states. The dynamic characteristics including natural periods and mode shapes of the system are calculated.

2. Platform description and mathematical model

2.1 Description of the platform

In this study, an oil platform that located in Gulf of Suez, was originally designed and built as a four-pile platform installed in approximately 78 m water depth and penetrated below mudline as shown in Fig. 2. It had one Boat landing and six barge bumpers, there were nine conductors and tree risers connected by the platform. The top of air gap zone (wave-deck clearance) located at elevation (+6.52 m) with respect to Lowest Astronomical Tide; LAT. The platform consists of three parts as shown in Fig. 3.

First, Topside, formed from four decks (helideck at

Fig. 3 Photo of the studied platform at the site

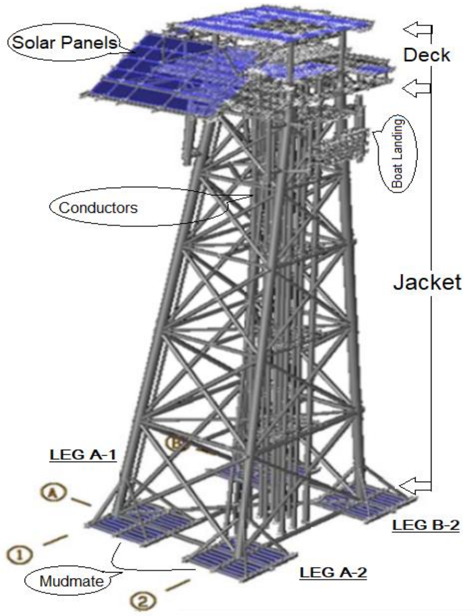


Fig. 4 3D finite element model of substructure and topside

elevation (+20.10 m), mezzanine deck at (+15.50 m), main deck at (+12.50 m) and cellar deck at (+ 8.70 m w.r.t. LAT).

Second, substructure, a jacket structure consists of four legs and six horizontal brace levels, top dimensions (plan at elevation + 5.00 m) are 10.34 m by 12.21 m and base dimensions on seabed (plan at elevation -77.985 m) are 22.586 m by 26.938 m.

Third, foundation, each of jacket legs is supported by a single pile, which extends along the main leg line, below the mud line, up to the pile penetration depth. The pile penetration depth is about (102 m). The pile has a tubular section with outer diameter of 48 inch (121.92 cm) and wall thickness of 2 inch (5.08 cm). The properties of the structural steel used in the platform are; density 7.85 t/m^3, Young's modulus 210 kN/mm^2, shear modulus 80 kN/mm^2, coefficient of thermal expansion 111.7 e-7/ C^o, Poisson's ratio 0.3, and material yield strength is equal to 345 MPa for thickness ≤ 40 mm and 335 MPa for thickness >40 mm.

2.2 Substructure and topside modelling

A 3D finite element model of the substructure and topside is prepared reflecting its in-place condition. This structural model includes all framing members represented correctly with its cross-sectional properties and their variations along with the appropriate lengths, joint eccentricities and the end connections. A detailed 3D model of the platform was carried out using SACS suite software (Bentley Systems 2011) which including jacket, deck, piles, stubs and supporting guides for conductors, risers and appurtenances as shown in Fig. 4. All members were modeled as 3D frame elements that are rigidly connected to each other. Shim plate centralizers inside the jacket leg at horizontal planes were simulated by dummy members restrained at the 6-dofs at jacket leg and restrained at two lateral dofs at pile end. Welding of pile to top of jacket leg was simulated by modeling both pile and jacket members rigidly connected to those joints. All conical transitions were modelled to account for the stress concentration around the cone joints. Helideck plating was modeled as membrane element to simulate its participation in the overall lateral stability. Solar panels were modeled by plates with zero weight and stiffness to consider wind loads acting upon them through applying proper overrides in the hydrodynamic model. Conductor guides and mudmat plating were modeled to calculate their weight and buoyancy. All jacket appurtenances like boat landing, risers, mudmats, barge bumpers and conductors were included in structural model to consider their associated loads and to check the jacket members and nodes where it is connected to those appurtenances. However, their participation in the stiffness of the structure was eliminated. The coordinate system is the right hand Cartesian system with the origin at the center of the deck legs and lies at LAT elevation, with +ve Z-axis vertically upward and the (+) ve X-axis pointing to the platform east then the (+) ve Y-axis determined using the right hand rule.

2.3 Miscellaneous and appurtenances modelling

All the jacket miscellaneous and appurtenance structures those are required to withstand the in-place loading conditions, are accurately covered in the finite element model with proper releases, such that their hydrodynamic and stiffness characteristics are truly represented. Major miscellaneous and appurtenances modelling items could be stated as: Caissons/J-Tubes are generally connected rigidly with main structures and are modelled as structural members. The density of the caisson pipe is factored to cover the weight of the internals of the drain caisson. Boat Landing/Protector Structure model is included in the global analysis. The effect of rub strip and the shielding of the members are accounted for hydrodynamic modelling. It is customary to model boat landing/protector structure as dummy sub-structure elements to generate environmental loading and then exclude these elements from the stiffness analysis. The conductors above mudline are self-standing, their weight is not transferred to any part of the structure, are modelled as structural members. These conductor pipes are connected to the structure at the horizontal framing levels where guides are provided through wishbone connections. These wishbone connections simulate the effect of the guide, releasing these sections axially and at the same time transferring the lateral loading to the adjacent framing. As all the conductors are installed along with the jacket, conductors were modeled as piles to a depth of 50 m below mudline, with identical stiffness properties. Such an idealization permits the conductors to share the global lateral loads and permits the guide framing to be designed to withstand the appropriate loading.

Risers are modelled either as a dummy or an appurtenance structure to account for their contribution to the environmental loads. SACS do not include the appurtenance mass in dynamic analysis and hence it is recommended to model risers as dummy structure. Risers are usually supported on hanger clamps at the jacket walkway level and on guide clamps at all other levels. The vertical load of the riser is thus transferred at the hanger clamp location and lateral loads are transferred at the guide clamp locations. Appropriate member releases are assigned to the riser connecting stubs to simulate this load transfer.

In case of plated connection for conductor guide framing, an equivalent framing member simulating the plated connection is provided with an override on the member diameter to represent the correct environmental loading on the plated connections. These members are assumed weightless, the weight of the conductor guides is separately assessed and input as joint loading at the appropriate nodes. The environmental loading on anodes (non-modelled) is included by globally increasing the drag and inertia coefficients by 5% to 7%. The weight of anodes is put in as joint loading at the appropriate nodes.

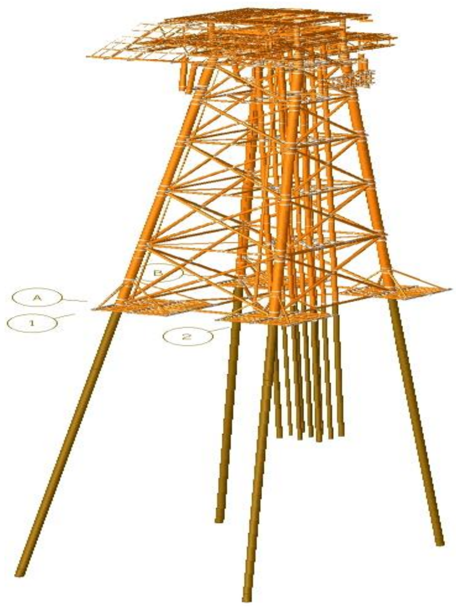


Fig. 5 Piles are enclosed inside the jacket leg

3. Soil-pile-jacket interaction modelling

3.1 Simulation of soil-pile interaction

The objectives of the specified geotechnical site survey fieldworks were the identification of the underlying soils and determine their geotechnical properties to enable the engineering analysis in connection with piled foundations for a platform structure. These values were used to generate the pile axial adhesion, skin friction and bearing capacity based on API-RP2A recommendations (DNV 1981, API 2014). Soil basic properties at the site were also used to generate the pile lateral soil properties in the form of load deflection curves. The modelling of foundation piles and conductor piles is constructed based on the pile/conductor size and penetration as defined in the design drawings. The foundation is simulated in the structural model by considering the pile stiffness; the lateral behavior of the soil and the nonlinear pile-soil-interaction. Soil properties for the site were obtained based on geotechnical investigations and bore hole data at the platform site.

The soil conditions are modelled as a set of nonlinear springs. Geotechnical data in the form of soil lateral capacities (P-y), axial values (T-z) and end bearing values (Q-z) curves are obtained from the soil and foundation report (Abdel Raheem and Hayashikawa 2013). Group effect for the piles and conductors shall be calculated if the center to center spacing is less than 8 times the diameter of the piles/conductors. As all the conductors are installed along with the jacket, conductors were modeled as piles to a depth of 50 m below mudline. However, to simulate the reality that the conductors or some of them may not exist during some duration of the platform life, the reactions of the conductors were checked to assure they don't exceed the hydrodynamic loads that those conductors attract and according the conductor don't share in the resisting the shear load on the platform. Iterative analysis was carried out by pile soil interaction (PSI) program till reaching the pile head displacement and rotation convergence. Thereafter, PSI extracts the final pile head loads and analyzes the pile. Being non-linear, the analysis was carried out for the combination load as basic load cases. This was achieved by passing the load combination generated by SEASTATE program to PSI program (Bentley Systems 2011) as basic load cases. The interface joints between the linear structure and the nonlinear foundation must be designated in the SACS model by specifying the support condition 'PILEHD' on the appropriate JOINT input line.

3.2 Simulation of pile-jacket interaction

For substructures with the space between the pile and jacket not grouted, the interaction of the piles inside the jacket leg was modelled using wishbone connections. Wishbone member was simulated in SACS by a fictitious member connecting the jacket node to the pile node, rigid offsets were specified to make the wishbone orientation same as the jacket leg at the pile end of the wishbone member; member end conditions are specified to release all the rotational degrees of freedom and the axial translation.

This model represents reasonably the interaction between the main pile and leg shims. Since the piles are enclosed inside the jacket leg, wave load contribution on the piles and wishbones were set to zero by giving the member dimension overrides. Piles and legs were considered flooded for in-place analysis, as shown in Fig. 5.

3.3 Soil profile and geotechnical characterization

Pile foundations are an essential structural component of jacket-type offshore platforms, and the pile soil interaction is of great concern in structural behavior. Table 1 shows the soil profile configurations at the platform site. The soil parameters are given in terms of Submerged unit weight (γ'), Undrained shear strength (S_u), Soil-pile friction angle (δ) and Over consolidation ratio (OCR) for piled foundation analyses are presented as a design soil profile for the study borehole location at east south of Gulf of Suez.

4. Hydrodynamic modelling

A rough type marine growth was considered in the analysis as from elevation (+2 m) to (- 15 m) with respect to Mean Sea Level (MSL) is 50 mm thick, from elevation (-15 m) to (-50 m) with respect to MSL is 25 mm thick and from (-50 m) to seabed with respect to MSL is 00 mm thick. The density of the marine growth was input as 1308 kg/m^3 rather than 1400 kg/m^3 in order to skip the contingency over the marine growth weight. This approach was derived by the fact that SACS considers marine growth as part of the structural weight, thus the application of a contingency on the structural weight affects marine growth weight as well. Drag and inertia coefficients for tubular members (C_d and C_m) were taken as 0.683 and 1.68 for smooth surface, and 1.103 and 1.26 for rough surface, respectively. Drag and inertia coefficients were magnified by 5% to account for the unmodelled anodes.

5. Structural loading

In general load cases for gravitational loads, the load cases considered in the analysis consist of jacket self-weight and jacket appurtenances weight, buoyancy loads, wave and current loads, curved conductor reactions, berthing/mooring loads, topside loads and wind loads. The self-weight of all jacket structural members in the model is generated by the SACS - SEASTATE program module using member cross sectional areas and materials densities. The dry weights of the modeled items and marine growth are as displayed in Table 2. Weight of un-modeled items like anodes, grating, handrail, etc. were obtained from the weight control report of the jacket and topside which input as joint and/or member loads in separate load conditions. Values for weight of key un-modeled structural elements are tabulated in the Table 3. Care is exercised through different contingency factors for the primary steel weight as (Jacket bracings, support structure of risers, conductor guides, main girders, module trusses, deck legs for helideck, main girders, helideck trusses) and secondary steel weight as (boat landings, bumpers, mudmats, secondary girders with depths <400 mm, deck plating, secondary members of the helideck structure). Live loads were modeled in accordance with the structural design basis. Open area live loads where imposed on members applying simple pressure load of 1 kN/m^2 intensity in the basic load cases thus allowing live load to be properly factored in the design combinations. To account for the area reserved by equipment footprints (skid/pressure loads) with negative values were used. The live loads used for the different design cases are summarized in Table 4. The total blanket live loads considered in the analysis as shown in Table 5. Equipment (including both itemized and bulks) dry and content weights were obtained from the weight control report (Gross weights rather than net weights were used to enable applying separate contingency for each equipment) and input as joint and/or member loads. The jacket legs, piles, caissons, J-tubes are considered flooded from mudline to MSL. Conductors and risers are modelled as non-flooded members. Conductors and riser content dry weight are calculated and explicitly applied as loads on the members. Remaining jacket tubular members are considered buoyant. Buoyancy acting on un-modeled items below MSL was also calculated and input in the same manner as self-weight of un-modelled items. The buoyancy forces for all the design waves are calculated employing the marine method in SACS. In order to allow the application of contingencies on the dead weight only, (and not on the buoyancy) the dead weight are generated two times first by

Table 1 Soil properties for the study borehole location

Top Depth (m)	Bottom Depth (m)	Soil type	γ' (KN/m^3)	S_u (kPa) from	S_u (kPa) to	OCR	Soil-Pile Friction Angle
0.0	1.4	Sand	6.4	-	-	-	20
1.4	5.6	Clay	6.9	35	70	20.0	-
5.6	8.8	Sand	8.3	-	-	-	20
8.8	10.5	Sand	8.3	-	-	-	15
10.5	14.5	Clay	9.3	60	85	3.6	-
14.5	15.9	Sand	7.4	-	-	-	15
15.9	18.0	Clay	7.4	120	120	6.3	-
18.0	20.0	Sand	7.4	-	-	-	15
20.0	21.0	Clay	7.4	120	120	4.9	-
21.0	24.8	Sand	8.8	-	-	-	30
24.8	31.7	Sand	5.4	-	-	-	15
31.7	34.3	Sand	6.9	-	-	-	20
34.3	39.5	Sand	8.0	-	-	-	15
39.5	44.8	Sand	7.4	-	-	-	15
44.8	46.1	Clay	8.8	250	250	3.7	-
46.1	51.5	Sand	7.0	-	-	-	15
51.5	56.5	Sand	8.9	-	-	-	25
56.5	60.5	Clay	10.3	300	330	3.0	-
60.5	68.0	Sand	9.1	-	-	-	15
68.0	70.5	Sand	9.1	-	-	-	20
70.5	75.0	Clay	10.3	270	300	2.0	-
75.0	79.5	Sand	9.3	-	-	-	15
79.5	88.5	Sand	9.3	-	-	-	15
88.5	97.4	Sand	9.3	-	-	-	20
97.4	105.4	Clay	10.3	400	440	2.1	-
105.4	106.4	Sand	9.3	-	-	-	15

Table 2 Dry weights of the modeled items and marine growth

Item	Net Weight (ton)
Modeled Deck Structure	178
Modeled Jacket Structure	656
Above Mudline Piles	505
Appurtenances	
• Boat landing	21
• Barge Bumpers	41
• Risers	22
• Conductors	327
Marine Growth	183

Table 3 Values for weight of key un-modeled items

Description	Unit Load	Total Net Weight (KN)
FRP Grating	0.20 kN/m^2	108.72
Steel Grating	0.50 KN/m^2	60.17
10 mm Plating	0.77 KN/m^2	232.95
8 mm Plating	0.68 KN/m^2	104.03
Deck handrails	0.19 KN/m`	82.71
Jacket handrails	0.16 KN/m`	5.752

Table 4 Live loads used for the different design cases

Area, UDL (KN/m^2)	Flooring & Stringers	Main Deck Girders	Main Truss Framing	Substructure
Laydown and Storage Areas	20	15	10	5*
Stairways, access platforms, walkway	5	2.5	2.5	-
Helideck	25	15	10	3*
Open areas	5	5	5	2.5

considering the normal water depth (buoyancy load is considered) and next with the water depth equal to 0.0 m (so that no buoyancy is created). Later, when load cases are combined into combinations the dead weight without buoyancy is used to represent the weight contingencies on self-weight only.

Provisions and requirements of the American Petroleum Institute (API 1993, 2010) and the project basis of design introduced six environmental loading conditions in Table 6. Wind, wave and current are assumed to act concurrently in the same direction. Eight loading directions were considered as two end-on directions 0° and 180°, two broadside directions 90° and 270° and four perpendicular to jacket diagonal directions 40°, 140°, 220° and 320°. The Omni directional wave parameters: wave height and actual period were taken from the metaocean criteria. Doppler effect of the current on wave was accounted for by calculating the apparent period for all the considered waves. SEASTATE program is used to calculate the apparent period based on the actual wave period, water depth and current velocity. Two-dimensional wave kinematic were determined from the stream wave theory for the specified wave height, water depth, and apparent period. The stream function order was automatically determined by SEASTATE. Wave kinematics factor was taken equal to 0.866. A series of wave stepping runs was carried through

Table 5 Blanket total live loads

Total live load	Weight, T
Substructure design (max. vertical load)	223.27
Deck truss design	591.15
Deck main girder design	748.05
Deck floor beams design	1078.8

Table 6 Environmental loading conditions

Condition	Return period (year)			Water Depth, m
	Wind	Wave	Current	
Operating Storm with min. water depth	1	1	1	77.58
Operating Storm with max. water depth	1	1	1	79.88
Extreme Storm-1 with min. water depth	100	100	10	77.29
Extreme Storm-1 with max. water depth	100	100	10	79.99
Extreme Storm-2 with min. water depth	10	10	100	77.44
Extreme Storm-2 with max. water depth	10	10	100	79.93

Table 7 Dynamic characteristic of the studied offshore platform

Mode	Freq.(cps)	Gen. Mass	Eigenvalue	Period (sec)
1	0.334	2244.7	0.2274	2.996
2	0.405	2337.9	0.1543	2.468
3	0.956	2862.5	0.0277	1.046
4	1.268	1676.5	0.0157	0.788
5	1.275	1435.4	0.0156	0.784
6	1.963	572.9	0.0066	0.509
7	2.154	365.7	0.0055	0.464
8	2.364	103.2	0.0045	0.423
9	2.530	29.1	0.0040	0.395

the structure to achieve the maximum overturning moment for the diagonal wave or base shear for the perpendicular and parallel waves. The Omni directional current profiles were taken from the metaocean criteria for offshore platform position. Profiles were nonlinearly stretched up to wave crests. Current blockage factors were taken as 0.80 and 0.85 for end-on/ broadside directions and diagonal directions, respectively. The increase in forces on the structure due to its dynamic response to the environmental loading was accounted for by applying the appropriate Dynamic Amplification Factor (DAF) on wave basic load cases based on the results of the dynamic analysis. For the wind, the Omni directional 1-hour mean wind speeds were extracted from the metaocean criteria and used for analysis of the substructure (jacket structure). Omni directional 1-minute mean wind speeds were extracted from the metaocean criteria and used for analysis of the top structure (deck structure). Flat wind areas were generated for wind loads imposed on equipment/bulks installed on the deck levels.

Orthogonal and diagonal wave directions are analyzed for the in-place condition. The water particle velocities and accelerations for the design waves are computed using a suitable wave theory which chosen by SACS. Current and wave directions are assumed collinear, the resultant particle velocities being the vector sum of these components. SACS calculate drag and inertia forces on individual members using Morison's equation. The wind loads on the topside facilities are computed externally considering the wind speed, shape of the structure, solidity ratio and its elevation with respect to the MSL. The wind speed is classified as: gusts that average less than one minute in duration, and sustained wind speeds that average one minute or longer in duration. The procedure adopted for force calculation is in conformance with API-RP-2A specification.

6. Methodology and numerical analysis

The procedure for reassessment of offshore platform for this study is referred to the standard AISC-ASD and API RP2A-WSD (AISC 2005, API 2014). In-place analysis is performed using SACS structural analysis computer program for different loads conditions. In-place analysis was performed by considering loading conditions for still water case, 1-year operation condition and 100-year extreme condition cases. Still water condition case combines maximum operation loads without considering the environmental load, while operational conditions using extreme environmental loads with 1-year return period, and for extreme conditions using extreme environmental loads with 100-year return period. Design and strength of structures are expressed in Unity Checks (UC) as the ratio between the actual stress that occurs on the member of structure with allowable stress. The numerical model of a case-study platform includes full soil–pile-structure interaction modelling. The jacket structures are designed to meet the requirement as stipulated in international (AISC 2005, Malley 2007). The design of the jacket structure that is studied complies with code requirement with enough robustness to withstand either in-service conditions or extreme conditions. The components of the platforms are analyzed under operating and under extreme storm conditions. The main difference between operating and extreme storm conditions is the wave height, current velocity, wind speed and wave period. The day-to-day operating and extreme storm environmental criteria are used to assess the respective structural response of the platform structures. The operating case defines the occurrence of a sea condition, with the probability of at least once in everyone month while the storm/survival case is an extreme sea state condition with 10^{-2} probability of exceedance in one year. Both operating and extreme sea state (100-year return period) conditions must meet the standard requirements for the design and reassessment of fixed offshore structures (Abdel Aal 2012, Henry *et al*. 2017).

6.1 Dynamic characteristics for the studied platform

In order to acquire the dynamic characteristics of the platform, a modal analysis was performed using the DYNPAC module of the SACS package. It uses a set of master (retained) degrees of freedom, selected to cover intersection joints, to extract the Eigen values (periods) and Eigen vectors (mode shapes). All stiffness and mass properties associated with the slave (reduced) degrees of freedom are included in the Eigen extraction procedure. The stiffness matrix is reduced to the master’s degrees of freedom using standard matrix condensation methods. The mass matrix is reduced to the master’s degrees of freedom using the Guyan reduction method assuming that the stiffness and mass are distributed similarly. All degrees of freedom which are non-inertial (no mass value) must be slave degrees of freedom. After modes are extracted using the master’s degrees of freedom, they are expanded to include full 6 degrees of freedom for all joints in the structure. It was decided to extract and use the first 40 mode shapes in order to properly simulate the dynamic response of the platform. Mass was simulated as mass of modeled items, mass of un-modeled loads, marine growth mass, water add mass and entrapped water mass. The first three mode shapes are the dominant mode shapes correspond to sway, surge and torsion modes of the platform. First nine frequencies and natural periods based on the platform data and site foundation characteristic were shown in Table 7. Mode shapes that the platform will vibrate in free motion, dominate the motion of the platform during environmental excitation, as presented in Fig. 6. The results of numerical analysis show that soil-pile-structure interaction (SPSI) decreases natural frequency of structure and increases equivalent modal damping of the structure. The effects of SPSI are significantly illustrated at higher modes compared to first mode for all dynamic characteristics.

6.2 Characterization of the in-place analysis

Design of offshore structures involves a variety of subjects including effects of harsh environmental conditions, variable loading patterns at each stage of the work and different accident scenarios. In-place analysis of an existing offshore module for the relevant actions in order to verify structural capacity and ensure safety of the structure. In the design of platforms one of the crucial loading parameters is the direction of wave loading on piles. For the analysis it is necessary that the waves be incident on the platform from differing directions to achieve the condition of critical loading direction. The in-place analysis is performed for studying platform subjected to 72 different load combinations cases within three main storm conditions, called as operation storm, extreme storm-1 and extreme storm-2 conditions as shown in Fig. 7. The main factors which drive and control the different storm conditions are the environmental loads return periods and the water depth variation

6.3 Characterization pile-soil interaction responses

Soil–structure interaction plays an important role in the behavior of structure under static or dynamic loading. It influences the behavior of soil, as well as the response of pile under loading. The analysis is highly essential for predicting a more accurate structural behavior so as to improve the safety of structures under extreme loading

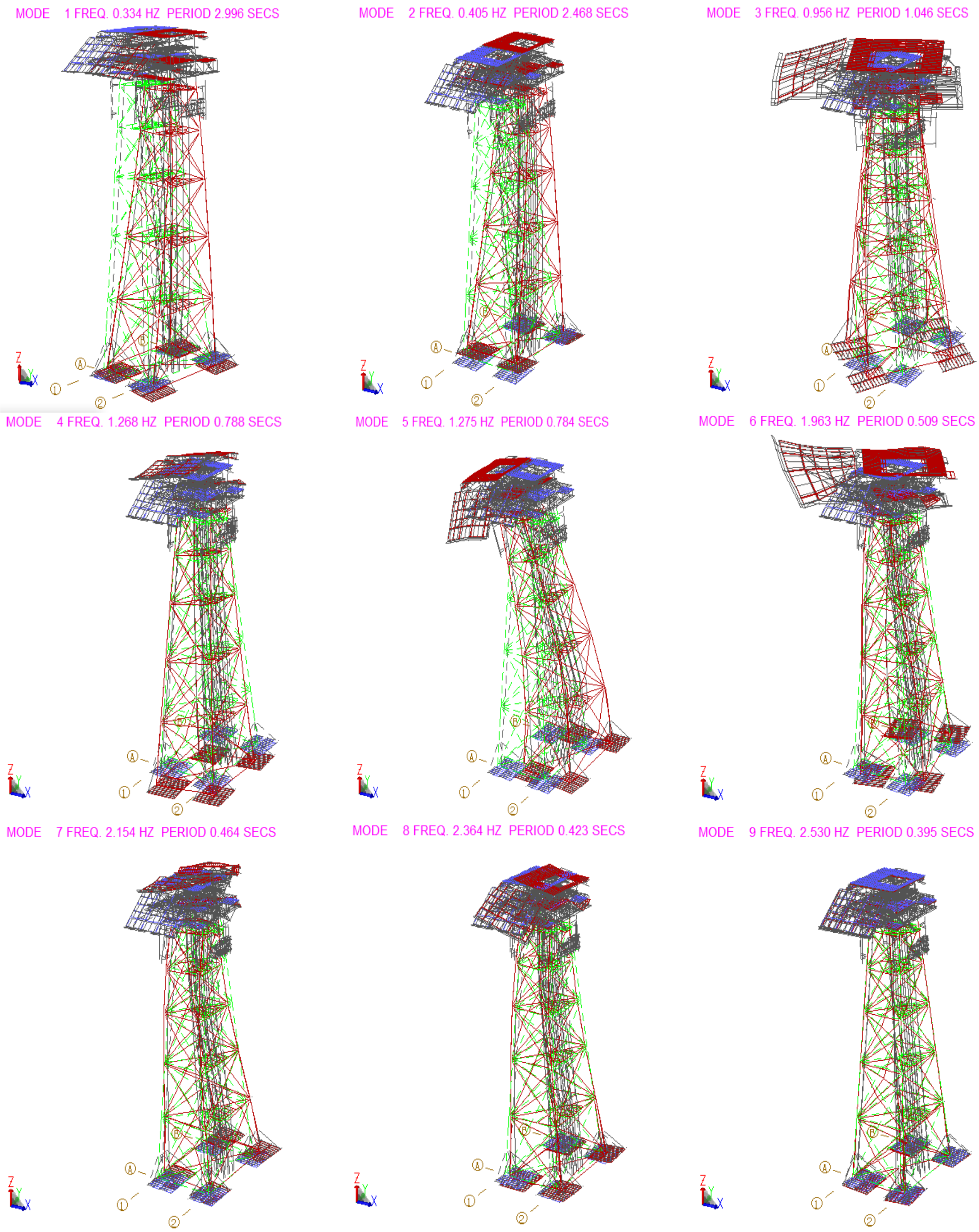
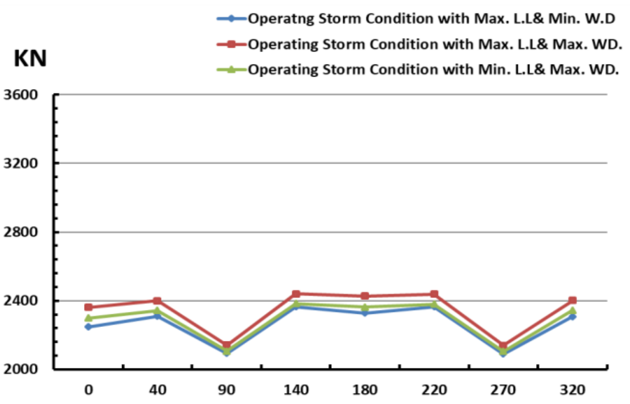
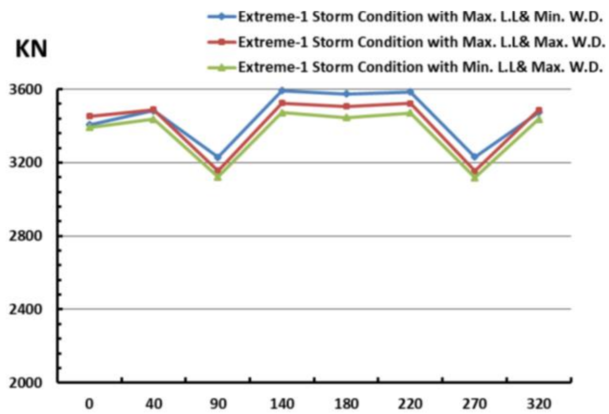


Fig. 6 First nine mode shapes and natural frequencies of the platform at site

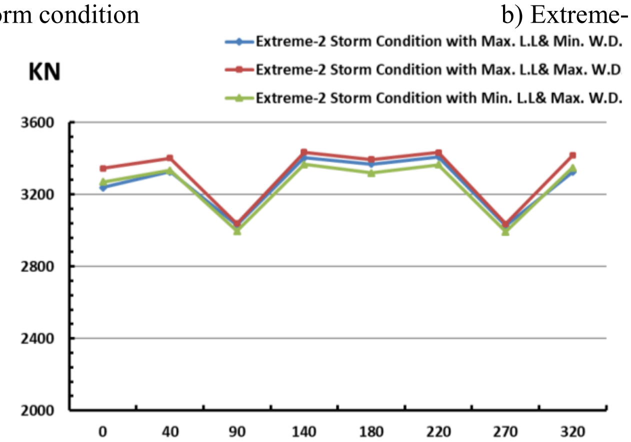
conditions. It is essential for a fixed offshore platform supported by pile foundation to resist lateral loading due to wind, wave and current forces acting on the platform, since minimal movement is required to provide stable workplace. Thus, the design of the pile foundation should satisfy the complicated and uncertain environmental load conditions. The response of the pile foundation to the environmental load is strongly affected by pile structure and soil structure interaction. There is a significant interaction between the jacket structure, its foundation piles, and the soil. The pile-



a) Operating storm condition



b) Extreme-1 storm condition



c) Extreme-2 storm condition

Fig. 7 Total applied horizontal loads for different incidence angles of the environmental loads' direction

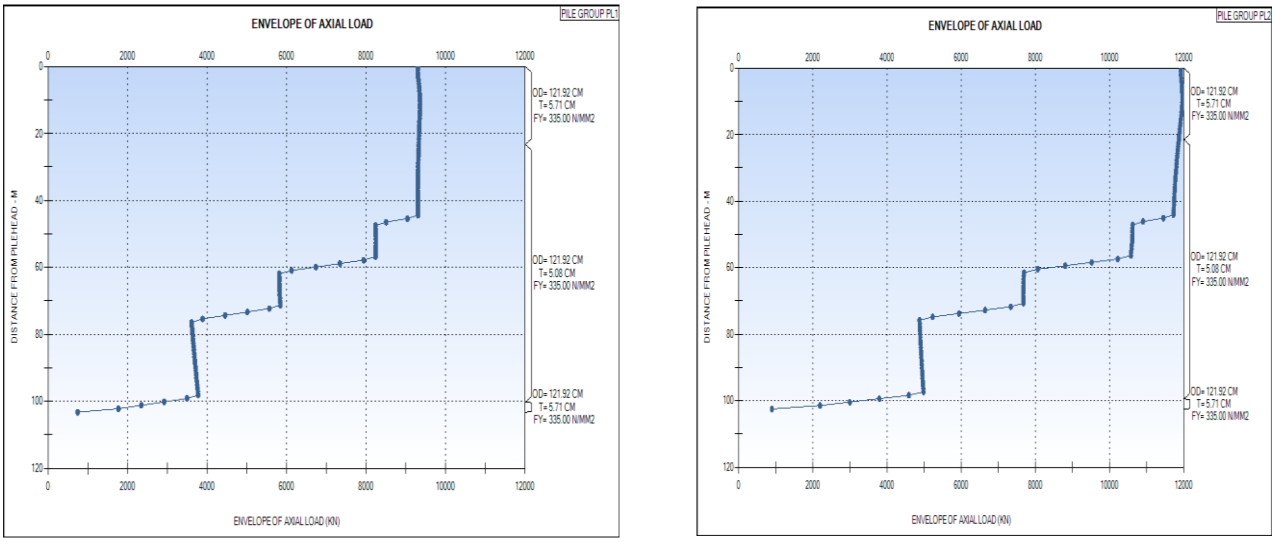
soil interactions results are displayed as envelope for all load combinations cases for pile axial force response, pile bending moment, axial deformation, lateral displacement and soil resistant. The analysis results help in the assessment of the platform structure with soil-pile interaction under in-place analysis due to the different combinations of the environmental storm conditions and gravity loads

6.3.1 Pile axial force response demand

The envelopes of axial force response of all piles are inversely proportional with penetration depth, where the axial forces decrease as the penetration depth increases, start with large value at the pile head then decrease on stages until the final penetration level according to soil layers capacities. The envelope of axial force responses of all piles have similar pattern, but the response values of the two piles that located in east direction of platform (PL-2 and PL-4) are larger than that for (PL-1and PL-3) in west side of platform. Fig. 8 illustrates the envelope of axial force response for piles in conjunction with penetration depth from pile head level under all load combinations cases for all storm conditions. The highest axial load values for pile (PL-1) between the pile head and the pile tip (level -103.3 m from pile head level) is equal to 9352.71 KN at penetration depth 11.36 m, and the pile PL-3 has axial force response smaller than that of PL-1, the highest axial load value for pile (PL-3) is equal to 9218.32 KN at the same penetration depth. On the other hand, the piles in the other direction of platform PL-2 and PL-4 have highest axial load values among the pile head and the pile tip (level -102.51 m from pile head level) equal to 11952.59 KN and 11877.81 KN respectively at penetration depth 9.23 m from pile head level.

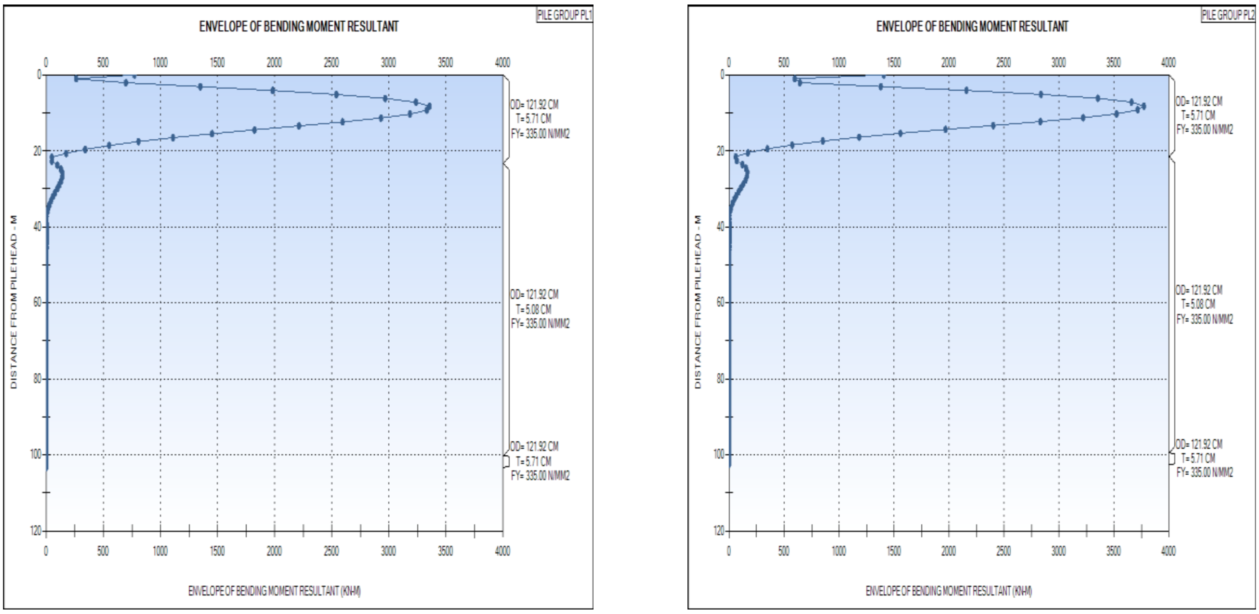
6.3.2 Pile bending moment response demand

The envelope of bending moment resultant (about y- and z-directions) for all load combinations cases of all piles are nonlinear proportional with penetration depth, where the absolute bending moment response increases as the penetration depth increases up to a shallow depth around 8.26 m below pile head level, then start a decrease trend up a penetration depth level of 22 m below pile head. The envelope of bending moment resultant of all piles has a similar pattern, but the response values of the two piles that located in east direction of platform (PL-2 and PL-4) are larger than the (PL-1and PL-3) in west side of platform. Fig. 9 illustrates the envelope of bending moment resultant in conjunction with penetration depth from seabed under all load combinations cases for all storm conditions. The highest bending moment resultant values for pile (PL-1) between the pile head and the pile tip (level -103.3 m from pile head level) is 3353.66 KN. m at penetration depth 8.26 m from pile head level. The pile PL-3 has a little bit smaller response value than that of PL-1, the maximum bending moment response demand for pile (PL-3) is equal



(a) Pile Group 1 (PL-1) (b) Pile Group 2 (PL-2)

Fig. 8 Envelope of axial load of different Load cases for piles group vs depth from PILEHEAD level



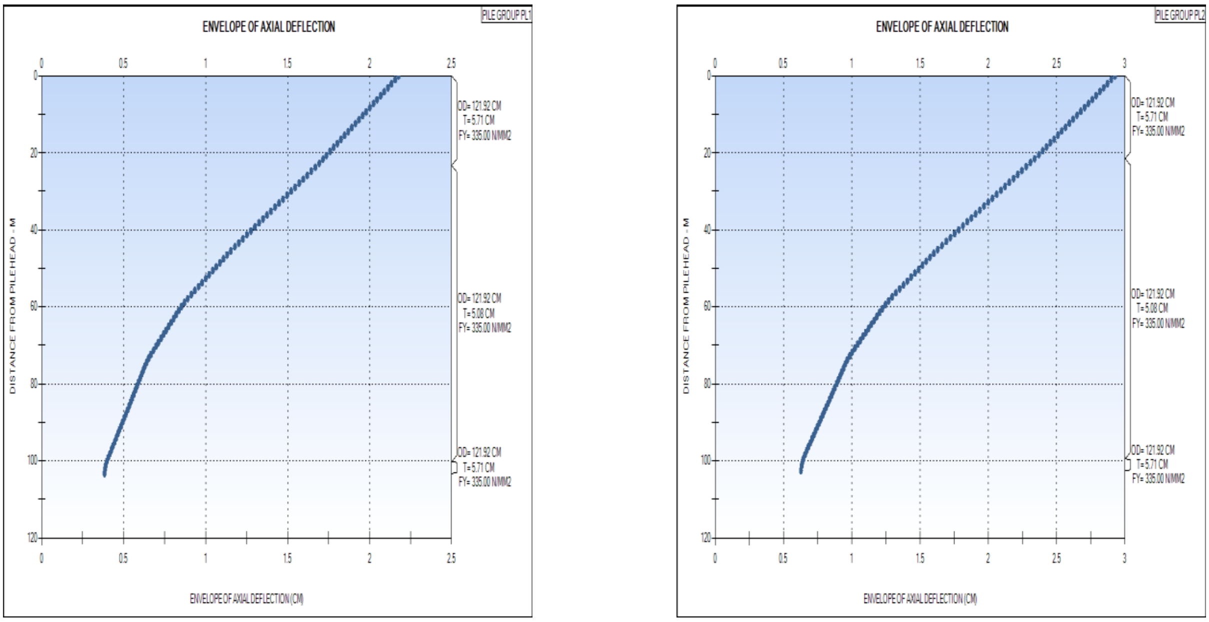
(a) Pile Group 1 (PL-1) (b) Pile Group 2 (PL-2)

Fig. 9 Envelope of bending moment resultant for piles groups vs depth from PILEHEAD level

3303.54 KN. m at the same penetration dept. On the other hand, the piles in the other direction of platform PL-2 and PL-4 have highest bending moment resultant values between the pile head and the pile tip (level -102.51 m from pile head level) equal to 3770.15 KN. m and 3702.77 KN. m respectively at the same penetration depth of other side 8.26 m. The absolute bending moment responses of all piles of platform are almost vanished after penetration depth 38 m from pile head level. The piles have the maximum absolute bending moment responses at the first fourth of pile penetration depth from pile head level. The pile heads subjected to concentrated bending moment resultants then drop approximately to third of the value in one-meter depth at west side of platform and drop to half of the value in one-meter depth at east side of the platform. Moreover, the influence of the soil-structure interaction on the response of the jacket foundation confirms that the flexible foundation model is necessary to estimate the loads of the offshore structure well.

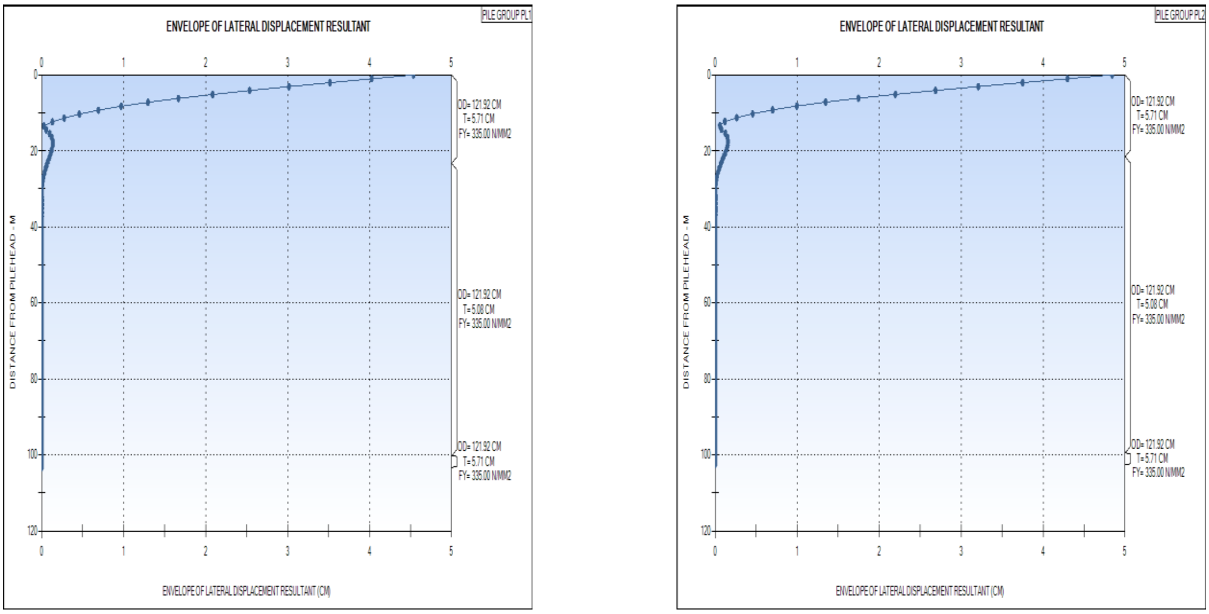
6.3.3 Pile axial deformation response demand

The envelope of axial deformation under all load combinations cases of all piles are inversely proportional with penetration depth, where the envelope of axial deformation response decreases as the penetration depth increases, starting with the highest response at the pile head then decrease up to the final penetration level according to soil layers capacities. The envelope of axial deformation response of all piles has similar pattern, but the values of the two piles that located in east direction of platform (PL-2



(a) Pile Group 1 (PL-1) (b) Pile Group 2 (PL-2)

Fig. 10 Envelope of axial deformation for piles groups vs depth from PILEHEAD level



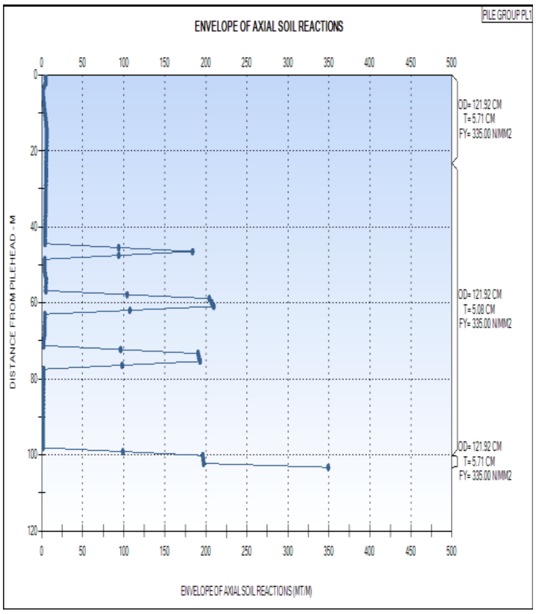
(a) Pile Group 1 (PL-1) (b) Pile Group 2 (PL-2)

Fig. 11 Envelope of lateral displacement resultant for piles groups vs depth from PILEHEAD level

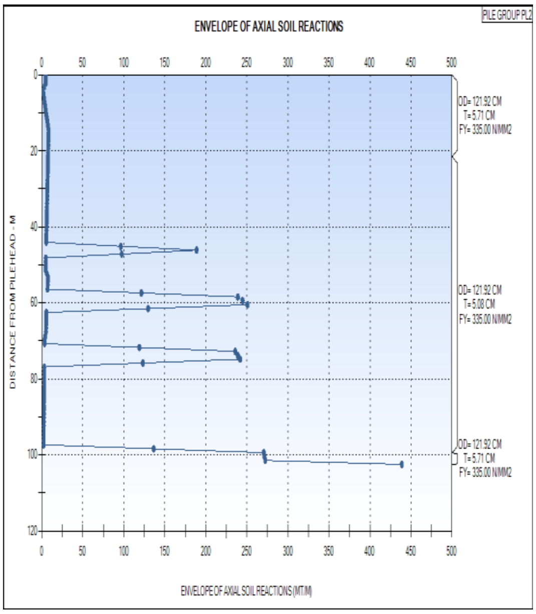
and PL-4) are larger than that of the (PL-1 and PL-3) in west side of platform. Fig. 10 illustrates envelope of axial deformation responses for piles in conjunction with penetration depth from seabed under all load combinations cases for all storm conditions. The highest axial deformation response values for all piles between the pile head and the pile tip (level -103.3 m from pile head level) are less than 3 cm.

6.3.4 Pile lateral displacement response demand

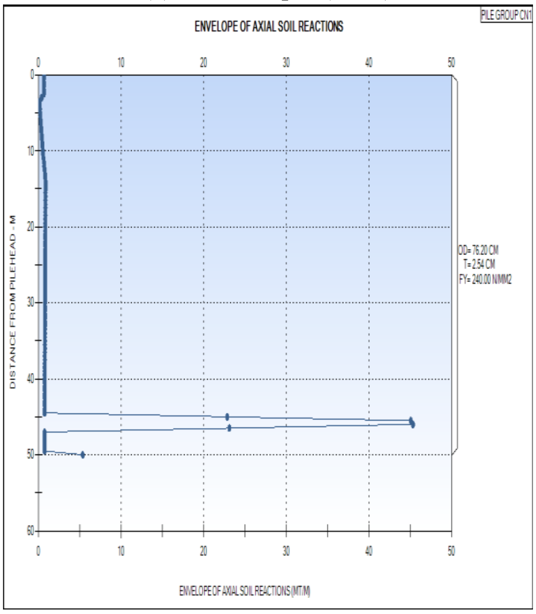
The envelope of absolute lateral displacement (resultant of y- and z-directions response) for all load combinations cases of all piles, are inversely proportional with penetration depth, where the lateral displacement responses decrease as the penetration depth increases until depth around 13.43 m below pile head level. Then start a bit little change as increase in the penetration depth until depth around 23.75 m below pile head level. Fig. 11 illustrates the envelope of lateral displacement resultant in conjunction with penetration depth from seabed under all load combinations cases for all storm conditions. The highest values for all piles are located at pile head level. The envelopes of lateral displacement resultants of all piles of platform are almost vanished after penetration depth 23.75 m from pile head level.



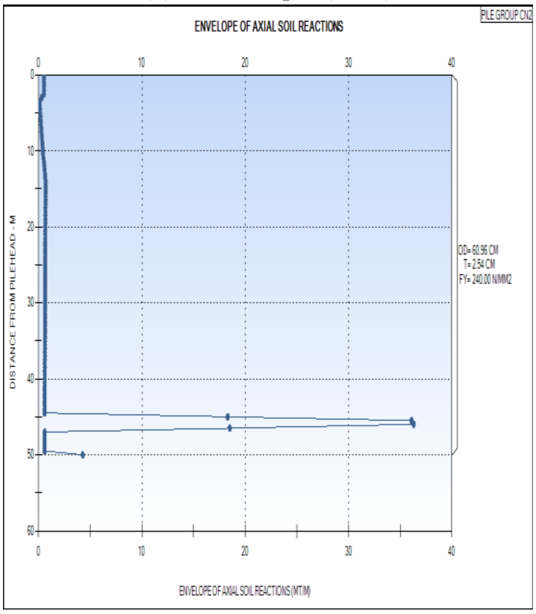
(a) Pile Group 1 (PL-1)



(b) Pile Group 2 (PL-2)



(c) Pile Group 5 (CN-1)

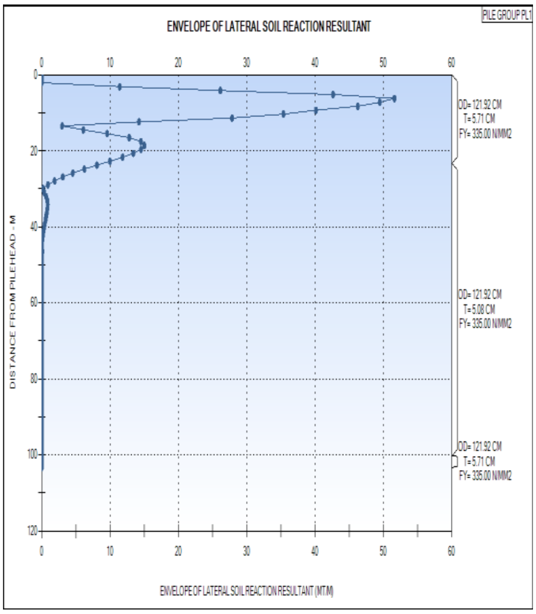


(d) Pile Group 6 (CN-2)

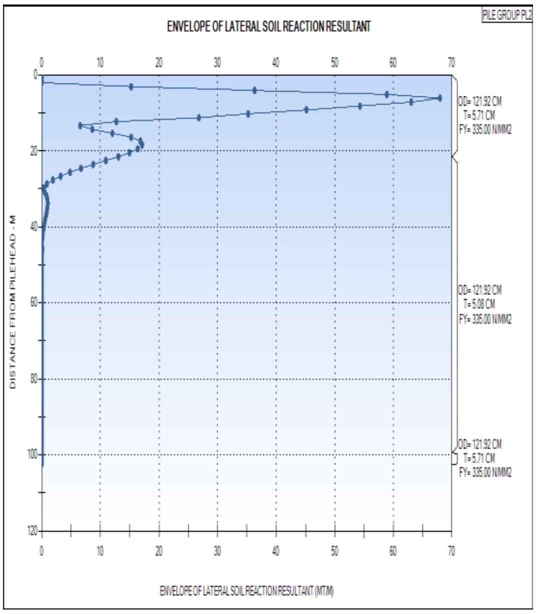
Fig. 12 Envelope of axial soil reaction for piles groups vs depth from PILEHEAD level

6.3.5 Soil axial reaction response

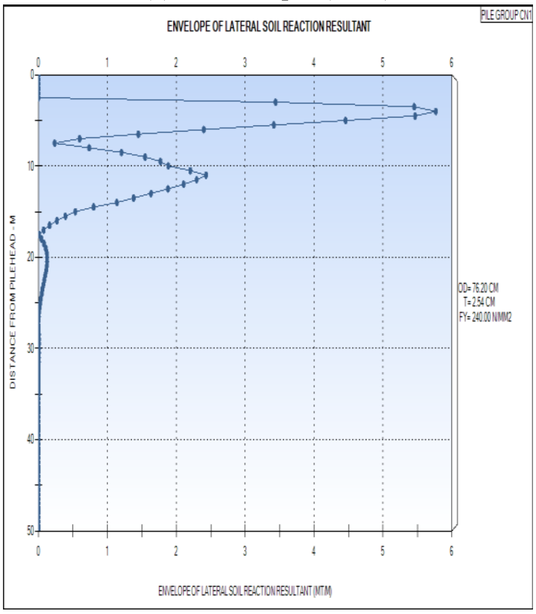
The envelopes of axial soil reaction response of all piles are nonlinear proportional with penetration depth. The axial soil reaction response has a small constant value as the penetration depth increases until around level 45 m under pile head level then increase suddenly to higher value through the soft soil layer after that return to a small constant value. This sharp change occurs four times along the total penetration depth and related the soil condition of soft soil layers. The largest values are shown at the pile tips levels (the maximum penetration level). The envelope of axial soil reaction of all piles are similar in configuration shape, but the values of the axial soil reaction surround the two piles that located in east direction of platform (PL-2/ PL-4) are larger than that of (PL-1/ PL-3) in west side of platform. Fig. 12 illustrates envelope of axial soil reaction responses by metric tons per meter long (MT/m) for piles in conjunction with penetration depth (m) from pile head level under all load combinations cases for all storm conditions. The highest axial soil response values around pile (PL-1) between the pile head and the pile tip level (-103.3 m from pile head level) is 438.81 (MT/m) at pile tip level and the pile axial soil reaction surround PL-3 has value smaller than PL-1 which the highest axial soil reaction values around pile (PL-3) equal 345.07 (MT/m) at the same depth as soil around PL-1. On the other hand the soil surround piles in the other direction of platform PL-2 and PL-4 have highest axial soil reaction values between the pile head level and the pile tip level (-102.51 m from pile head level) equal to 438.81 and 435.75 (MT/m) respectively at pile tip level. For



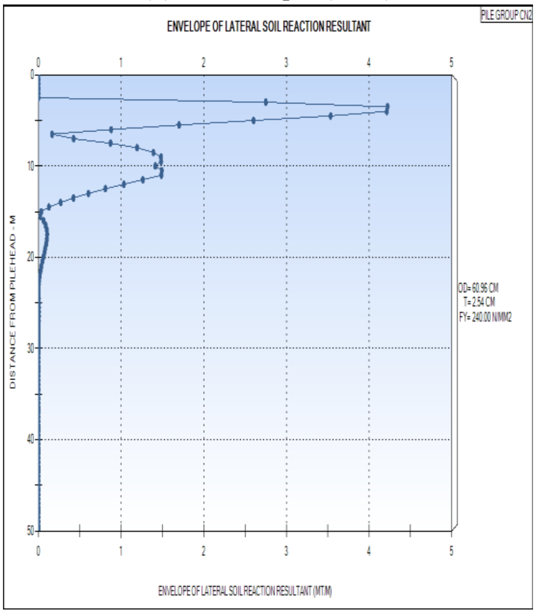
(a) Pile Group 1 (PL-1)



(b) Pile Group 2 (PL-2)



(c) Pile Group 5 (CN-1)



(d) Pile Group 6 (CN-2)

Fig. 13 Envelope of lateral soil reaction resultant for piles groups vs depth from PILEHEAD level

the axial soil reactions belong to casings of wells as its work as pile until penetration depth 50 m from piles heads levels, are the same behavior as the piles and the values of axial soil reaction around pile group CN-1 is a little bit greater than CN-2. Fig. 12(c) and 12(d) illustrates envelope of axial soil reactions for casing piles in conjunction with penetration depth from pile head level to level 50 m from pile head under all load combinations cases for all storm conditions.

6.3.6 Soil lateral reaction response

The envelope of lateral soil reactions resultant (from y- and z- directions) for all load combinations cases of all piles have nonlinear behavior and proportional with penetration depth. The lateral soil reactions resultant increase as the penetration depth increases until a shallow depth around 6.20 m below pile head level, then start decrease as the increase of penetration depth until around level 13.40 m, then start increase again until depth around level 18.50 m and decrease again until approximately vanish at depth level around 30 m to pile tips level. The envelope of lateral soil reactions resultant of all piles are similar in configuration shape, but the values of lateral soil reactions around the two piles that located in east direction of platform (PL-2 and PL-4) are larger than lateral soil reaction around piles (PL-1 and PL-3) in west side of platform.

Fig. 13 illustrates the envelope of lateral soil reactions resultant by metric tons per meter long (MT/m) for piles in conjunction with penetration depth from piles heads under all load combinations cases for all storm conditions. The highest lateral soil reactions resultant values around pile (PL-1) between the pile head and the pile tip (level -103.3 m from pile head level) is 51.58 (MT/m) at penetration depth 6.20 m and the lateral soil reactions resultant values around pile (PL-3) has a little bit value smaller than PL-1 which the highest lateral soil reaction values around pile (PL-3) equal 50.60 (MT/m). On the other hand the soil around piles in the other direction of platform PL-2 and PL-4 have highest lateral soil reaction resultant values between the pile head and the pile tip (level -102.51m from pile head level) equal to 67.94 and 66.49 (MT/m) respectively at the same penetration depth of other side 6.20 m. The lateral soil reactions resultants of all piles of platform are approximately vanished after penetration depth 30 m from pile head level. The maximum lateral soil reactions are in the first third of pile penetration depth. For the lateral soil reactions resultant surrounding casings of wells as its work as pile until penetration depth 50 m from piles heads levels, are the same behavior as the lateral soil reactions resultant surrounding piles and the values of lateral soil reactions resultant surrounding CN-1 is a little bit greater than lateral soil reactions resultant surrounding CN-2. Fig. 13(c) and 13(d) illustrates envelope of lateral soil reactions resultant for soil around casing in conjunction with penetration depth from pile head level to level 50 m from pile head under all load combinations cases for all storm conditions. It is customary to design piles that will allow initially low values of unity checks to add extra levels of safety to the design and for the longevity of the platform.

7. Conclusions

The in-place performance of the offshore platform is assessed using a finite element method using structural analysis computer system (SACS). The in-place analysis was performed for a typical offshore platform with a piled jacket-support structure under 72 different load combinations with three different storm conditions: operation storm, extreme storm-1 and extreme storm-2 conditions. The variation of the main factors of storm conditions: environmental loads return periods and the water depth is investigated for the response's demands of the different components of piled jacket-supported offshore platform. The results show that the studied platform has adequate strength capacity that can resist environmental loads safely, the drift demand of platform is within the allowable drift. The direction of environmental loads and water depth have significant effects on the results of in-place analysis. The in-place response investigation is quite crucial for safe design and operation of offshore platform.

The results of in-place numerical analysis show that soil-pile-structure interaction increases the natural frequency and equivalent modal damping of the structure. The effects of soil-pile-structure interaction are significantly illustrated at higher modes compared to first mode for all dynamic characteristics. Soil response shows a nonlinear behavior at the vicinity of seabed and the level of nonlinearity decreases dramatically as soil depth increases. The axial force response of all piles is inversely proportional with penetration depth, where it shows maximum response at the pile head and decrease as the penetration depth increases. The absolute bending moment response of all piles are nonlinear and are proportional with penetration depth, it has maximum response value through the first fourth of pile penetration depth. The axial deformations of all piles are inversely proportional with penetration depth, and it has the highest value at the pile head, then decreases as the penetration depth increases. The maximum lateral soil reaction responses are in the first third of pile penetration depth and approximately vanished after that end of penetration depth.

Pile-soil-jacket interaction increases the response along the pile length and alters the response of the platform base. It has a significant effect on the stresses along the pile shaft especially the bending moment, one of the most important parameters in the design. The properties of the soil layers have an important effect on the response of the platform and supporting piles, a decrease in the resistance of the upper soil layers results in an increase in the deformation response demands at the platform base and along the pile/jacket and a decrease in the shear force and bending moment response demands along the pile. The location of the maximum bending moment depends on the soil resistance of soil layers. The influence of the soil-structure interaction on the response of the jacket foundation confirms that the flexible foundation model is necessary to estimate the force and deformation demands of the offshore platform well. It is very difficult to design a structure which exactly matches a directional wave height distribution. Some adjustment of the directional criteria so that failure from waves in one directional band is more probable than another may thus result in a more optimal design and assessment of platform structure.

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