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Numerical analysis for structure-pile-fluid-soil interaction model of fixed offshore platform

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Abstract. In-place analysis for offshore platforms is required to make proper design for new structures and true assessment for existing structures. In addition, ensure the structural integrity of platforms components under the maximum and minimum operating loads and environmental conditions. In-place analysis was carried out to verify the robustness and capability of structural members with all appurtenances to support the applied loads in either operating condition or storm conditions. A nonlinear finite element analysis is adopted for the platform structure above the seabed and the pile–soil interaction to estimate the in-place behavior of a typical fixed offshore platform. The SACS software is utilized to calculate the natural frequencies of the model and to obtain the response of platform joints according to in-place analysis then the stresses at selected members, as well as their nodal displacements. The directions of environmental loads and water depth variations have an important effect on the results of the in-place analysis behavior. The influence of the soil-structure interaction on the response of the jacket foundation predicts is necessary to estimate the loads of the offshore platform well and real simulation of offshore foundation for the in-place analysis. The result of the study shows that the in-place response investigation is quite crucial for safe design and operation of offshore platform against the variation of environmental loads.

Keywords: FEM; offshore platform; storm conditions; 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

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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 2016, 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, Elsayed *et al*. 2015, 2016).

Offshore structures should be designed for severe environmental loads and strict requirements should set for the optimum performance (Abdel Raheem *et al*. 2012, 2013). Design calculations for offshore structures require a mathematical model which is based upon the state of the art in offshore technology. 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. Typically jackets have natural periods in the first mode ranging from 2 to 3 seconds. The wave period is typically between 6 to 10 seconds. 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). SACS is used to study the behavior of aged offshore jacket platforms based on pushover analysis (George *et al*. 2016). SACS software was used for global analysis of multi-directional wave loads for the jacket platform for fatigue evaluation of the composite non-tubular joint structure of an offshore jacket subjected to wave loads (Bao and Feng 2011). A dynamic spectral fatigue analysis using SACS has then been conducted to evaluate the remaining life of the platform in its in-place condition (Zeinoddini *et al*. 2016). A comparison of experimental and numerical dynamic responses of a prototype jacket offshore platform for both hinges based, and pile supported boundary conditions and soil-piles structure interaction is studied using SACS (Asgarian *et al*. 2012).

This paper represents a case study of an existing fixed offshore platform located in Gulf of Suez by in-place strength analysis. 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 modelling of offshore platform structure which includes the top side platform and the support structure is elaborated including aspects of structure modelling, piled structures, and hydrodynamic loading. Specifications of offshore structure model and environmental parameters for the site location of platform under consideration are determined. 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 a

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

steel jacket under combined structural and environmental loadings. Wave plus current kinematics are generated using wave theory. The horizontal components of the wave velocity and acceleration fields are multiplied by a wave kinematics factor that is intended to account for direction spreading 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. Natural periods and mode shapes of the system are calculated.

2. Platform description and modelling

2.1 Description of the platform structure

In this study, an oil platform that located in block 404 of Gulf of Suez, Egypt (Fig. 2), was originally designed and built as a four-pile platform installed in approximately 78 m water depth and penetrated below mudline. It has one Boat landing and 6 barge bumpers. There were 9 conductors and 3 risers connected by the platform. The top of air gap zone (wave-deck clearance) locates at elevation (+6.52 m) with respect to (w.r.t) lowest astronomical tide (LAT). The splash zone area locates between (- 2.579 m) and (+4.421 m) w.r.t LAT. The platform consists of three parts as shown in Fig. 3. First, Topside, is formed from four decks (helideck at 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 template jacket structure consist of 4 legs and six horizontal brace levels (-77.984 m, -57 m, -38 m, -21 m, -7 m, +5 m) all w.r.t LAT, top dimensions (plan at elevation + 5.00 m w.r.t. LAT) are 10.34 m by 12.212 m and base dimensions on seabed (plan at elevation -77.985 m w.r.t. LAT) 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

Fig. 2 Map showing Location of the studied offshore platform

Fig. 3 Photos of the study platform at site

used in the platform (as in the structural design basis of the model platform) are; Density 7.85 t/m^3, Young's Modulus 210 kN/mm^2, Shear Modulus 80 kN/mm^2, Poisson's ratio 0.3, Coefficient of Thermal Expansion 0.0001117/ C^o and material yield strength is equal to 345 MPa for thickness $\leq$ 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 shall be prepared reflecting its in-place condition. This structural model shall include all framing members represented correctly with its cross-sectional properties including the sectional variations along with the appropriate lengths, joint eccentricities, and the end connections. A detailed 3D model of the platform was carried out using the SACS suite software (Bentley Systems 2011, Abdel Raheem *et al*. 2018) which including jacket,

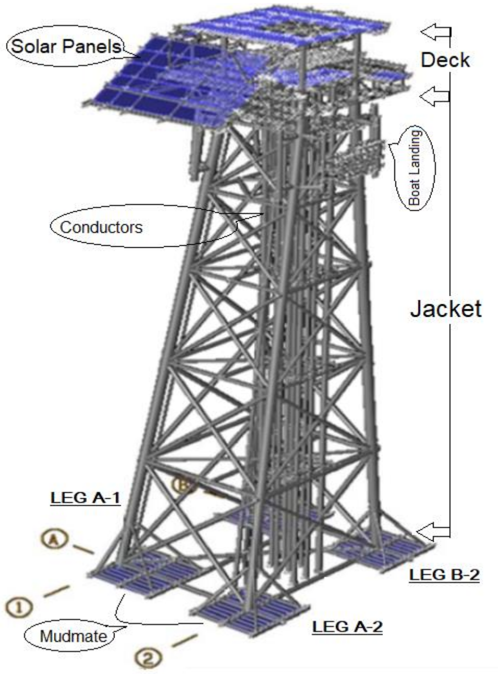


Fig. 4 3D finite element model

deck, piles, stubs and supporting guides for conductors, risers and appurtenances was used for analysis as 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 six DOFs at jacket leg and restrained at two laterals DOF at pile end. Welding of pile to top of jacket leg was simulated by modeling both pile and jacket members rigidly framing to those joints. All conical transitions shall be 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 by SACS program. 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

In general, all the jacket miscellaneous and appurtenance structures those are required to withstand the in-place loading conditions shall be accurately covered in the computer model with proper releases, such that their hydrodynamic and stiffness characteristics are truly represented. Major miscellaneous and appurtenances modelling items are explained below. 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. A fully idealized boat landing/protector structure model are included in the global analysis. The effect of rubstrip 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 anodes are projections on the surface of a tubular structural member, and they increase the hydrodynamic forces and hydrodynamic coefficients of the tubular member. The effect of anodes on hydrodynamic forces and coefficients depends on several factors. The industrial practice to account for this effect is to multiply the total hydrodynamic force by a global factor, which may be conservative and uneconomical. Examination and comparison of the various drag and inertia coefficient values shows little agreement on exact values. This is true for smooth cylinder values and even more so when a rough cylinder is involved. Differences of up to roughly 40% can be found when comparing the drag or inertia coefficients suggested by the various sources for a specific flow situation (Bhinder *et al.* 2015). The environmental loading on anodes (non-modelled) shall be included by globally increasing the drag and inertia coefficients by 5% to 7%. The weight of anodes is input as joint loading at the appropriate nodes. Alternatively, equivalent drag and inertia coefficients (Cd & Cm) override values may be calculated and assigned to the members. Drag and inertia coefficients are magnified by 5% to account for the unmodeled anodes. Member and group overrides that were entered into the in-place model to account for the environmental loads acting on the unmodeled items have been modified to account for the changes made to the drag and inertia coefficients. The modified Drag and inertia coefficients overrides used (El-Reedy 2015, Nallayarasu and Selvam 2020). Offshore structures such as jacket platforms have anodes fitted on their steel structural members to prevent corrosion.

3. Pile foundation soil interaction modelling

Pile foundations are an essential structural component of jacket-type offshore platforms, and the pile soil interaction is of great concern in structural behavior. The soil parameters based on geotechnical investigations and bore hole data at the platform site are determined in terms of Submerged unit weight (γ’), Undrained shear strength (Su), Soil-pile friction angle (δ) and Over consolidation ratio (OCR) for piled foundation analyses. 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 simulation of foundation in the structural model is performed by considering the pile stiffness, the lateral behavior of the soil and the nonlinear pile soil interaction. 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. 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. Static P-y curves for a single pile in sand can be established from the API guidelines.

$$p_u = min\begin{pmatrix}(C_1 \times h + C_2 \times D) \times \gamma' \times h \\ C_2 \times D \times \gamma' \times h\end{pmatrix} \tag{1}$$

p_u is the ultimate resistance, (kN/m); γ' is the effective soil unit weight, (kN/m3); h is the depth, (m); and D is average pile diameter from surface to depth h, (m). C_1, C_2, C_3 are coefficients determined from the API guidelines. The lateral soil resistance-deflection (p-y) relationships for sand are non-linear and may be approximated by the following expression:

$$P_s = A \times p_u \times \tanh\left(\frac{\kappa \times h}{A \times p_u} y\right) \tag{2}$$

where A is the factor to account for cyclic or static loading (A = 0.9 for cyclic, A = (3.0 − 0.8*(h/D)) ≥ 0.9 for static), p_u is the ultimate resistance at depth h, and κ is the initial modulus of the subgrade reaction determined from the API specifications. The vertical soil resistance along the pile shaft and at the pile toe is a function of the level and rate of loading. The soil resistance to the vertical movement of the pile is modelled using axial shear transfer functions that depend on local pile deflection (T–z curves). The soil resistance at the pile toe is modelled using Q–z curves.

Group effect for the piles and conductors are 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. 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, Abdel Raheem *et al*. 2020a, b) 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. For substructures with the space between the pile and jacket not grouted, the interaction of the piles inside the jacket leg are modelled using wishbone connections. Wishbone member simulation in SACS consists of a fictitious member connecting the jacket node to the pile node. At the pile end of the wishbone member offsets are specified to make the wishbone orientation same as the jacket leg. At the pile end of the wishbone, member end conditions are specified to release all the rotational degrees of freedom and the axial translation. This model represents reasonably the interaction between a main pile and leg shims. Since the piles are enclosed inside the jacket leg, wave load contribution on the piles and wishbones are set to zero by giving the member dimension overrides. Piles and legs are considered flooded for in-place analysis.

4. Hydrodynamic actions modelling

Marine growth accumulates on the legs and bracing with time. As a result, the diameter of the affected members will increase, so the lateral load due to waves will increase as well with time and can become critical if the marine growth thickness increases more than the predicted marine growth thickness in design. Therefore, removing marine growth can enhance structural capacity. A rough type marine growth of 50 mm was considered in the analysis for the platforms elements within elevation range from (+2 m) to (- 15 m) with respect to Mean Sea Level (MSL), and 25 mm within from elevation range from (-15 m) to (-50 m). The density of the marine growth was input as 1308 kg/m3 rather than 1400 kg/m3 in order not to consider a 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 will affect marine growth weight as well. Drag and inertia coefficients for tubular members were taken as Cd equal to 0.683 and Cm equal to 1.68 for smooth surface which for rough surface Cd and Cm equal to 1.103 and 1.26, respectively. Drag and inertia coefficients were magnified by 5% to account for the unmodelled anodes. Members in model override to apply for shielded members like piles and dummy members like members between piles and jacket legs (wishbones) to simulate the reality in-place position where those members do not attract environmental loads or have no buoyancy. Members were modeled as flooded or non-flooded as its position.

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 is generated two times first by 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.

5. Structural and environmental loads

In general load cases for gravitational loads, the individual basic load cases considered in the analysis shall 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 modelled are generated by the SACS - SEASTATE program module using member cross sectional areas and densities. The dry weights of the modeled items and Marine Growth are as displayed in Table 1. Weight of un-modeled items like anodes, grating, handrail, etc. were obtained from the weight control report of the jacket and topside of model platform 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 2.

Table 1 Dry weights of the modeled items and Marine Growth

No.	Item	Net Weight (ton)
1	Modeled Deck Structure	178
2	Modeled Jacket Structure	656
3	Above Mudline Piles	505
4	• Boat landing	21
	• Barge Bumpers	41
	• Risers	22
	• Conductors	327
5	Marine Growth	183

Table 2 Values for weight of key un-modeled items

Item	Description	Unit Load	Total Net Weight (KN)	Remarks
1	FRP Grating	0.20 kN/m^2	108.72	Fiber Reinforced Plastic
2	Steel Grating	0.50 KN/m^2	60.17	
3	10 mm Plating	0.77 KN/m^2	232.95	
4	8 mm Plating	0.677 KN/m^2	104.03	
5	Handrails	0.19 KN/m`	82.71	Deck handrails
		0.162 KN/m`	5.752	Jacket handrails

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/m2 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 3. The total blanket live loads considered in the analysis as shown in Table 4. Equipment (including both itemized and bulks) dry and content weights were obtained from the weight control report of model platform (Gross weights rather than net weights were used to enable applying separate contingency for each equipment) and input as joint and/or member loads.

Code provisions and requirements of the American Petroleum Institute (API 1993, 2010, 2014) and the project basis of design of six environmental loading conditions were introduced in Table 5. Wind, wave, and current are assumed to act concurrently in the same direction. Eight loading directions were considered as two end-on directions 0° & 180°, two broadside directions 90° & 270° and four perpendicular to jacket diagonal directions 40°, 140°, 220° & 320°. The Omni directional wave parameters (wave height H max & actual period TH max) 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 calculates 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 were carried through 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. 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 stream wave theory which chosen by SACS. Seastate generates the fluid particle velocities and accelerations at 740 points (37 stations along the wave and 20 elevations from the surface to the mudline). The horizontal locations are equally spaced from the crest to the trough. The vertical grid points are closely spaced near the surface, the spacing increasing arithmetically with depth. At every horizontal location, the vertical grid is defined by 20 points from the wave surface to the mudline - this defines a curvilinear grid. 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.

Table 3 live loads used for the different design cases considered in the analysis

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

Table 4 Total blanket live loads considered in the analysis

No.	Item	Weight (Ton)
1	Total Live Load for substructure Design (max. vertical load)	223.27
2	Total Live Load for Deck Truss Design	591.15
3	Total Live Load for Deck Main Girder Design	748.05
4	Total Live Load for Deck Floor Beams Design	1078.8

Table 5 Environmental loading conditions considered in the analysis

Condition	Return Period (Year)			Water Depth (w.r.t LAT) 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

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 may be 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. Analysis methodology and numerical results

The procedure for reassessment of offshore platform is referred to the standard AISC-ASD and API RP2A-WSD (AISC 2005, API 2014). In-place analysis is performed using structural analysis SACS program by considering all loads conditions. The loading conditions include: Still Water Condition, 1-year Condition and 100-year Condition. Still Water condition cases combine maximum operation load without considering the environmental load, while operational conditions using extreme environmental loads with return period 1-year, and for extreme conditions using extreme environmental loads with return period of 100-years. The unity check (UC) for strength is expressed as the ratio between the actual stress that occurs on the member of structure with allowable stress. The numerical modelling of the case-study platform includes full soil–pile-structure interaction modelling. The jacket structures are designed based on the code-based design method and to meet the requirements as stipulated in international standards (AISC, 2005, Malley 2007). The design of the jacket structure complies with code requirement with enough robustness to withstand either in-service condition or extreme condition. The components of the platform are analyzed under operating and under extreme storm conditions. The main difference between operating and extreme storm condition 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 structure. The operating case defines the occurrence of a sea condition, with the probability of at least once in every 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 (Henry *et al*. 2017).

6.1 Verification and validation assessment

Verification and validation are the essential procedures required to assess accuracy and credibility of numerical analyses. Verification is meant to identify and remove programming errors in a computer code and verify numerical algorithms. It deals with the mathematics, only, while the Validation is meant to assess the accuracy at which a numerical model represents reality and includes the essential features of a real model. Unlike verification, validation deals with the physics, Fig. 5. Generally, the end users do not need to verify commercially available software. It is the responsibility of the program developers to take care of the verification and ensure that their product is mathematically correct and is free of programming errors (so-called bugs). On the other hand, validation should be generally performed by the end user. It is the primary responsibility of the end user to create a numerical model that represents the real physical model by adopting appropriate boundary conditions, constitutive models, elements, entire numerical model. Validation of a finite element program is considered through the following (Brinkgreve and Engin 2013). The constitutive models and parameters are validated, where the capability of the model in simulating stress-strain responses is evaluated by modelling lab test results. In this process, soil parameters are iteratively varied to make a "best fit" to the test data. Boundary conditions validation is ensured, so that the selected boundaries do not influence the output of the analyses. Lateral and base boundaries are required to limit the extents of a finite element model and to optimize the analysis execution time. The meshing and spatial discretization are validated so that the finite element mesh is sufficiently fine so that the analysis outputs are remain about the same when a finer mesh is analyzed. After each of the model components is individually validated, the analysis outputs are validated for the entire numerical model by a comparison of the convergence of the numerical model with the reference results and an observation of the convergence of the model results. The Platform model was tuned to operate well on a reasonable spatial discretization and good mesh convergence study was done for an adequately refined mesh. This approach is tested for the problem introduced in the SACS manual and adopted the platform model in the current study.

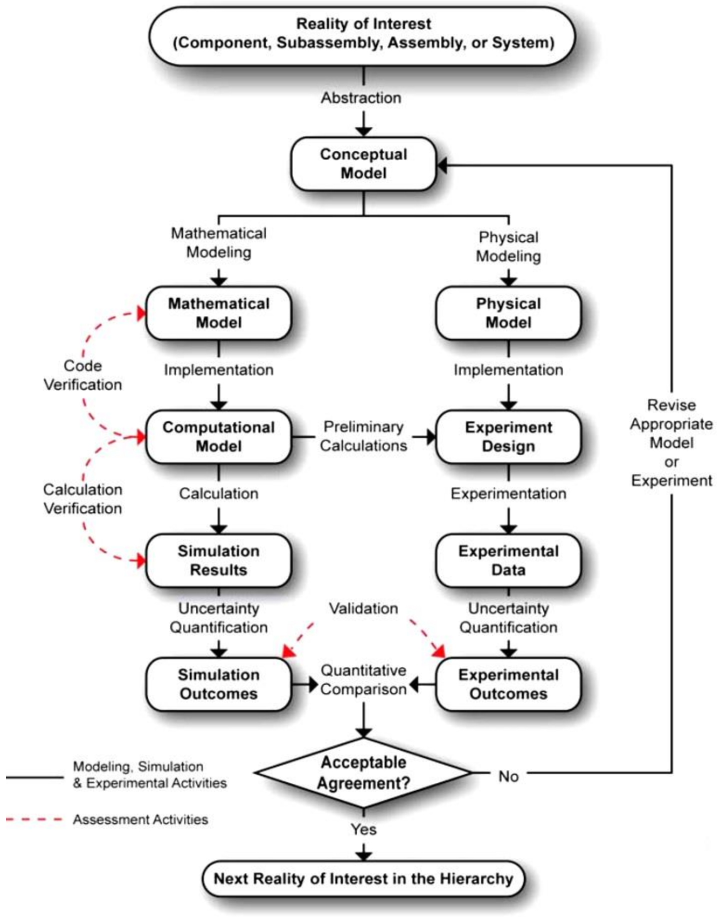


Fig. 5 Verification and Validation assessment through an entire analysis (ASME 2009, 2012)

Table 6 Dynamic characteristic of the offshore platform case study

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

6.2 Vibration characteristics for the platform

To acquire the dynamic characteristic of the platform, a modal analysis was performed using the DYNPAC module of the SACS package. A set of master retained degrees of freedom is 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. Mass was simulated as mass of modeled items, mass of un-modeled loads, marine growth mass, water added mass and entrapped water mass. Based on the mentioned structural specifications, a free vibration analysis was then carried out to generate the dynamic characteristics of the platform including vibration mode shapes and natural periods. The first 40 mode shapes are extracted when conducting the modal analysis to investigate the vibration characteristics of the studied platform, But The first dominant three vibration mode shapes are corresponded to sway, surge, and torsion modes of the platform. The cumulative mass of the first ten modes was found to be 99.89%, 99.84% & 99.31% for X, Y & Z directions respectively which are enough to represent the dynamic response of the platform in the earthquake spectral response analysis. First ten frequencies and natural periods, based on the platform data and site foundation characteristic, were calculated, and are shown in Table 6. Mode shapes represent the shape that the platform will vibrate in free motion and the shape dominates the motion of the platform during environmental excitation, are presented in Fig. 6. The first three modes of vibration are the primary interest as the first modes has the largest contribution to the platform motion during environmental excitation.

6.3 Characterization of the response demands

The in-place analysis of the studied platform is performed for 72 different load combinations of three main storm conditions: Operation storm, Extreme storm-1, and Extreme storm-2 conditions.

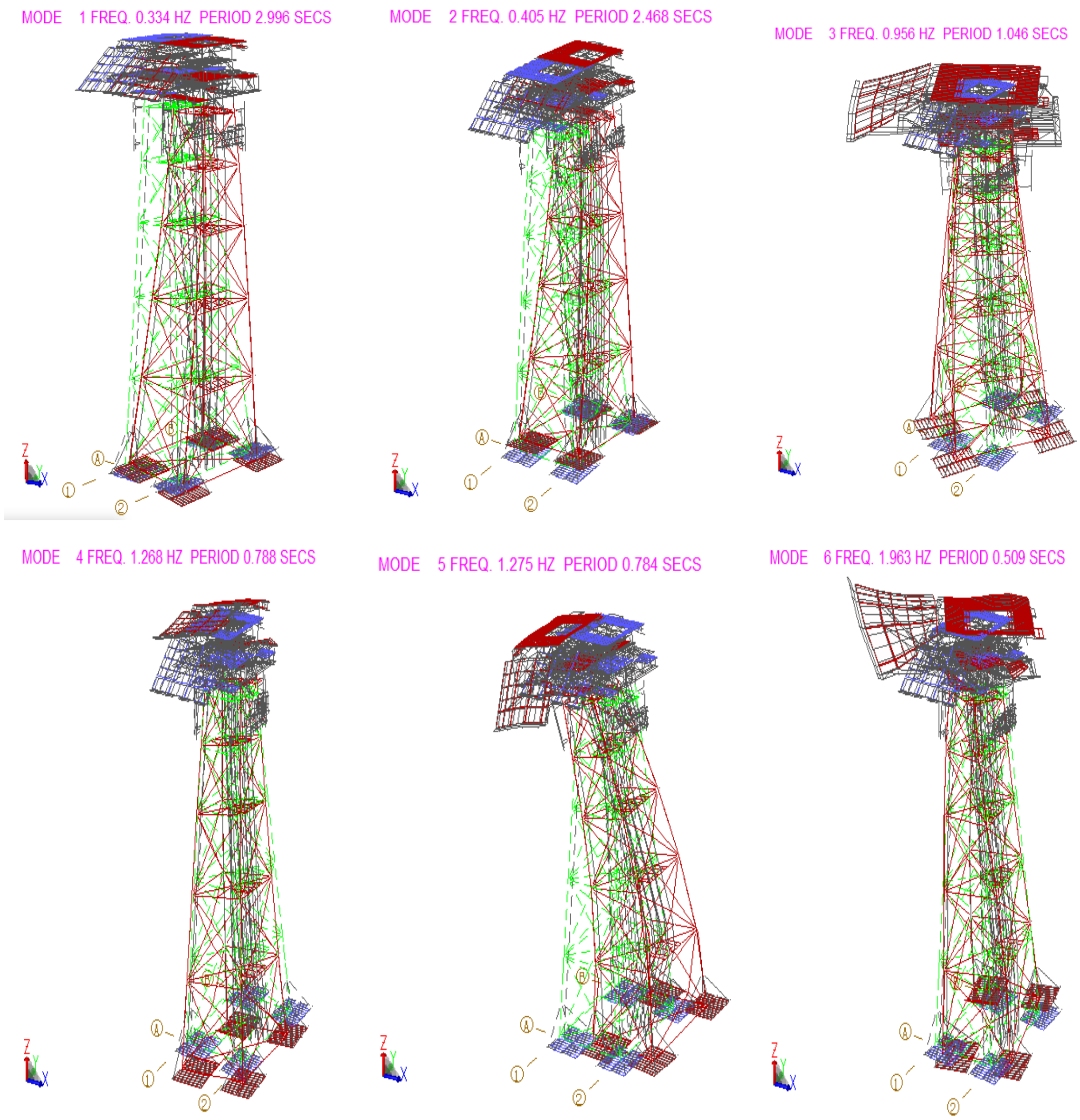
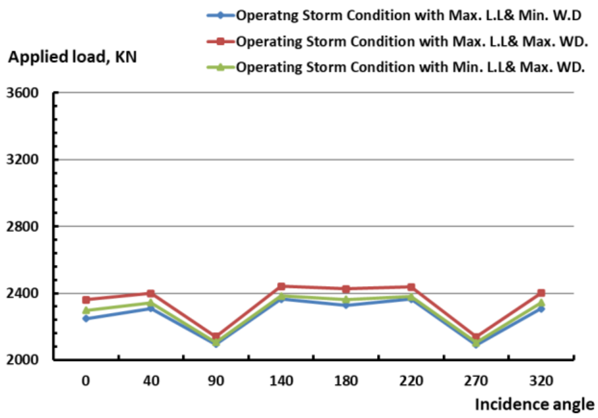
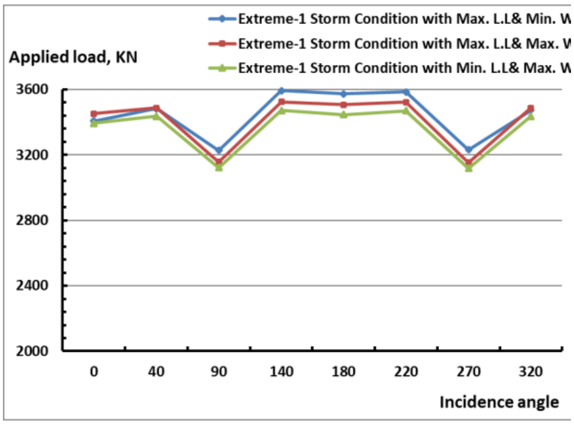


Fig. 6 First six mode shapes and natural frequencies of the platform at site

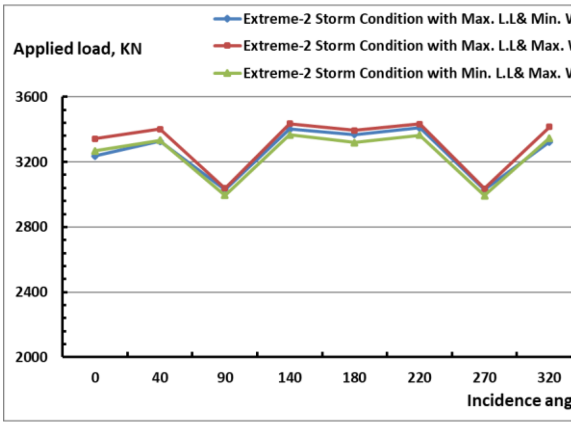
The main factors which drive and control the different storm conditions are the environmental loads return periods and the water depth variation. The outcome results in the study focus on main responses demands as base shear, overturning moment, and joints displacement that help in the assessment of the platform structure under in-place analysis.



(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

6.3.1 Base shear and overturning moment responses demands

Fig. 7 show the total applied horizontal loads that affect the platform for all load cases in different storm conditions and are used in the in-place analysis. The horizontal loads resultant in the operating storm condition is due to two variables: live load and water depth with respect to the incidence angles of the environmental loading direction. The three cases follow the same configuration as the maximum value displayed at perpendicular to jacket diagonal directions angles 140 & 220 degree and the minimum displayed at jacket broadside directions angles 90 & 270 degree but, the values of loads change in each case. The highest horizontal loads values were achieved by the operating storm condition with maximum live load and maximum water depth.

The Figs. 7 (b) and 7(c) shows the applied horizontal loads resultant in extreme-1 storm condition and extreme-2 storm condition, respectively. The two extreme storm conditions behaved like the operating storm condition with respect to different eight loading directions, but the highest horizontal loads value in extreme-1 storm condition accompanying with maximum live load and minimum water depth while, in the extreme-2 storm condition maintained the same trend as in operating storm condition. The water depth and live load play a vital role in the behavior of applied horizontal loads resultant on the offshore platform and the applied load values in the three environmental storm conditions, hence affecting the in-place analysis results of all responses including straining actions, displacement, velocity and acceleration. The three environmental storm conditions follow the same configuration while, their values are different. The live loads have an important role in vertical loading, where the minimum applied vertical loads get out from minimum live load and maximum water depth for all three environmental storm conditions. The water depth variations have no influence on the values of the vertical applied loads on the offshore platforms. Some of the important checks in design and analysis of offshore platform are base shear and overturning moment which the platform as a whole act and behave as a cantilever supported on seabed and extended through sea water until the designed height.

Table 7 summarize the maximum base shear and overturning moment acting on the platform due to environmental loading cases in different storm conditions that applied in the analysis. The in-place structural analysis of the jacket structure determines the structural response of the jacket due to environmental and gravity loads. The total environmental loading on the jacket structures is translated into overturning moment (OTM) and base shear (BS) at the mudline. The corresponding BS and OTM for different wave directions are investigated. The maximum BS and OTM occur when the wave attack angle is 140° except for the base shear under storm condition which occurs at 0° incidence angle. For these wave directions, the exposed surface area of the jacket is larger than any other directions and attract more wave and current loadings. In general, there are significant increases in the BS and OTM, the percentage increment of base shear ranges from 40.7% to 47.1% and the percentage increment of OTM ranges from 39.8% to 47.8%. This indicates that the jackets are wave dominated structures.

6.3.2 Joints displacement response demands

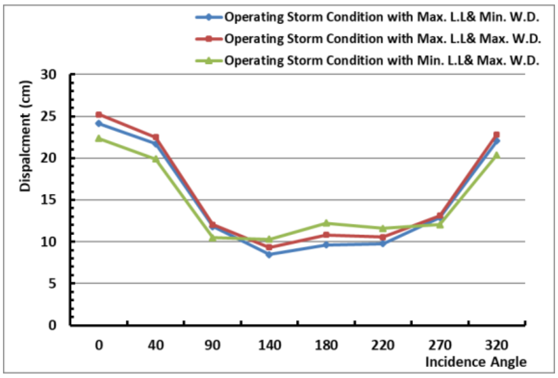
The joints displacement responses from platform analysis are very vital results to all or any risers, pipelines, rotating equipment, instruments, and all control devices connected and glued to platform. The values of joints displacement of the platform could influence the service function of all things that connected to platform and therefore the increasing of displacement more than the allowable limits not only cause harmful effect for platform structure but also for all connected items and devices which could lead to hazards and disasters for all area.

6.3.3 Horizontal displacement response demand

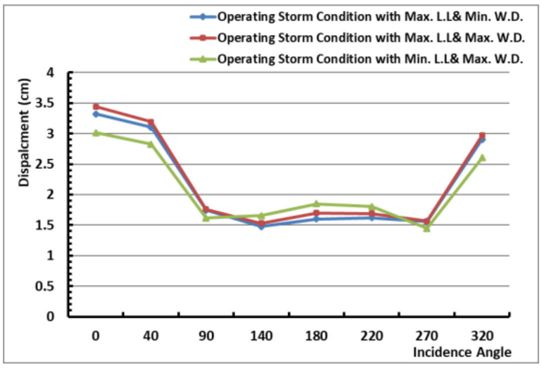
Horizontal displacement response of the offshore platform is one of the main important results from the in-place analysis and has strong relations with the environmental loads. Table 8 summarized maximum relative values (drift) among the mudline and the topmost deck for horizontal displacement of all platform legs according to the three storm conditions.

Table 7 Maximum base shear and overturning moment due to environmental conditions

Load Type	**Base Shear** (KN)	**Overturning Moment** (KN.M)	**Water Depth**	**Direction**
Operating Storm	2441.24	149592	Maximum	140°
Extreme Storm-1	3592.07	221142	Minimum	140°
Extreme Storm-2	3435.42	209108	Maximum	140°



LEG A-1 Top Level



Leg A-1 mudline level

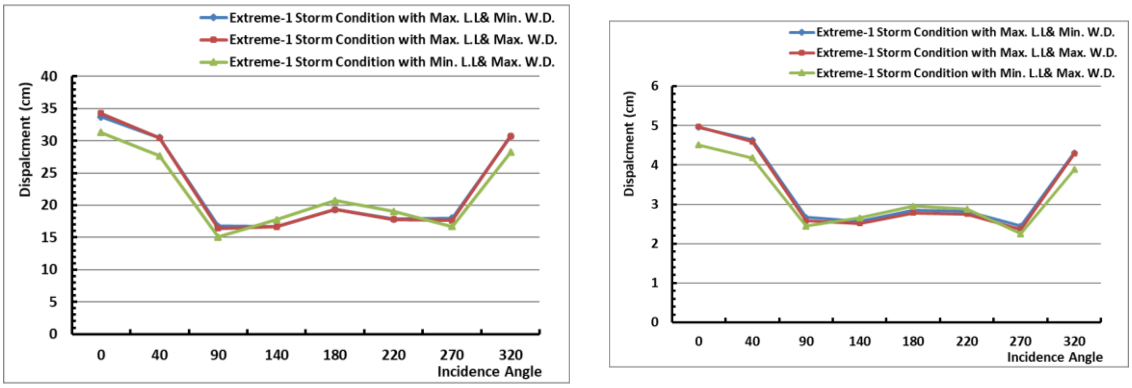
Fig. 8 Absolute horizontal displacement for operating storm condition

All drift is acceptable as the allowable drift for platform equal to height/200 = 49.05 cm. Figs. 8 -10 illustrated the absolute horizontal displacements for most top level and mudline level of one of the four legs (Leg A-1) for offshore platform under the three storm conditions (operating, extreme-1 and extreme-2 storm conditions) for different incidence angles of environmental storm conditions, water depth variations and live loads. Figures displayed that the platform legs have the same configuration of the absolute horizontal displacements in all storm conditions but different in values. The maximum value of horizontal displacements is accompanied with angle of environmental loads direction 0 degree then the two perpendicular to jacket diagonal directions (40 and 320 degree). In all storm conditions three load cases are studied with respect to live loads and water depth variations (maximum live load with minimum water depth, maximum live loads with maximum water depth and minimum live loads with maximum water depth).

The maximum live loads cases achieved maximum displacement values in all environmental directions except environmental loads directions 140, 180 and 220 degree in all platform legs. For the operating storm conditions, the absolute horizontal displacements are affected by variation of water depth with constant live loads with respect to environmental loads direction.

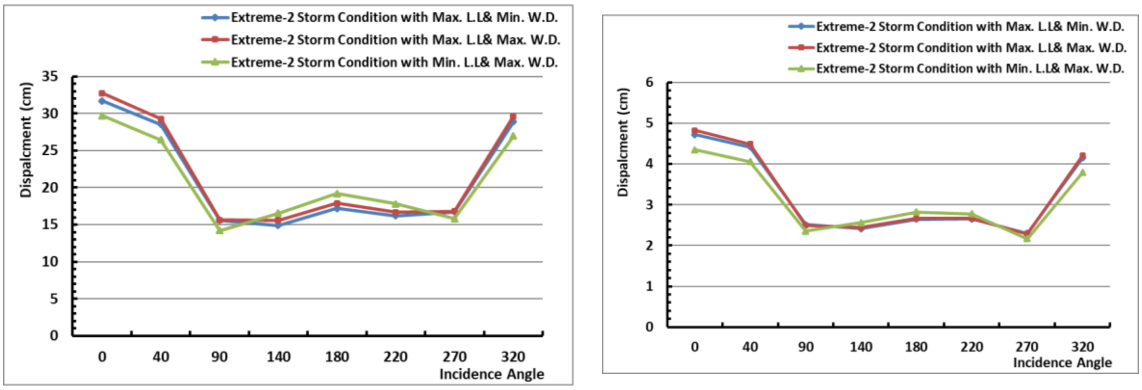
Table 8 Maximum relative horizontal displacement values (drift) for all platform legs

Leg	Levels	Maximum Absolute Values (cm)	Storm Condition	Relative Value (Drift) (cm)
A-1	Mudline (-78 m)	4.97	Extreme-1	29.28
	Most Top (+20.1 m)	34.25		
A-2	Mudline (-78 m)	4.96	Extreme-1	29.24
	Most Top (+20.1 m)	34.20		
B-1	Mudline (-78 m)	4.88	Extreme-1	29.30
	Most Top (+20.1 m)	34.18		
B-2	Mudline (-78 m)	4.87	Extreme-1	29.27
	Most Top (+20.1 m)	34.14		



LEG A-1 Top Level Leg A-1 mudline level

Fig. 9 Absolute horizontal displacement for extreme-1 storm condition



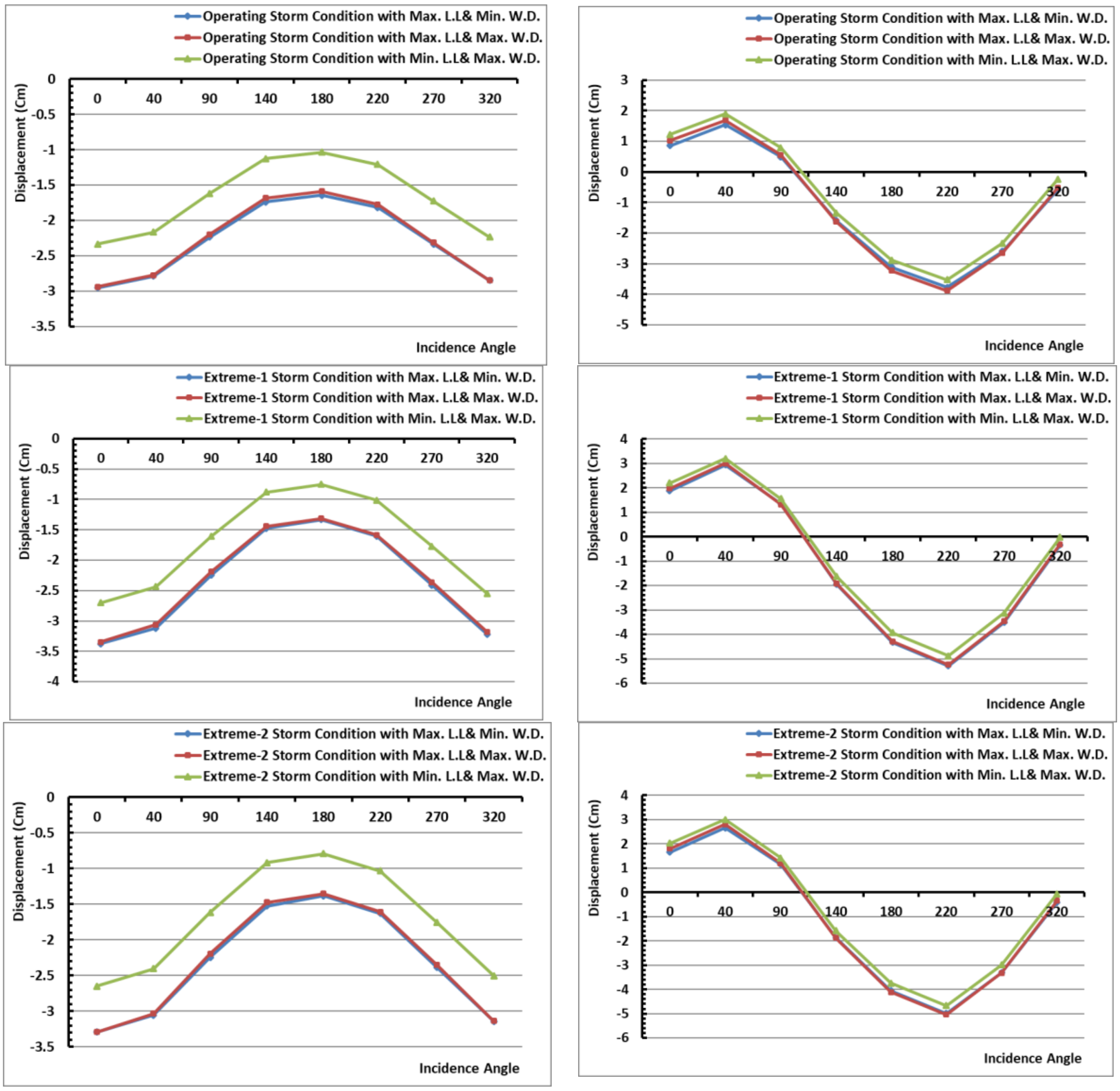
LEG A-1 Top Level Leg A-1 mudline level

Fig. 10 Absolute horizontal displacement for extreme-2storm condition

The absolute horizontal displacements in extreme-1 storm condition according to angles of environmental loads directions at top and mudline levels of all platform legs are investigated. The displacements values do not vary with variation of water depth. The maximum horizontal displacements between the mudline and the topmost deck in all platform legs for the three storm conditions result from load combination (wave, wind, and current loads in direction 0° with maximum water depth, dead load and maximum live load).

6.3.4 Vertical displacement response demand

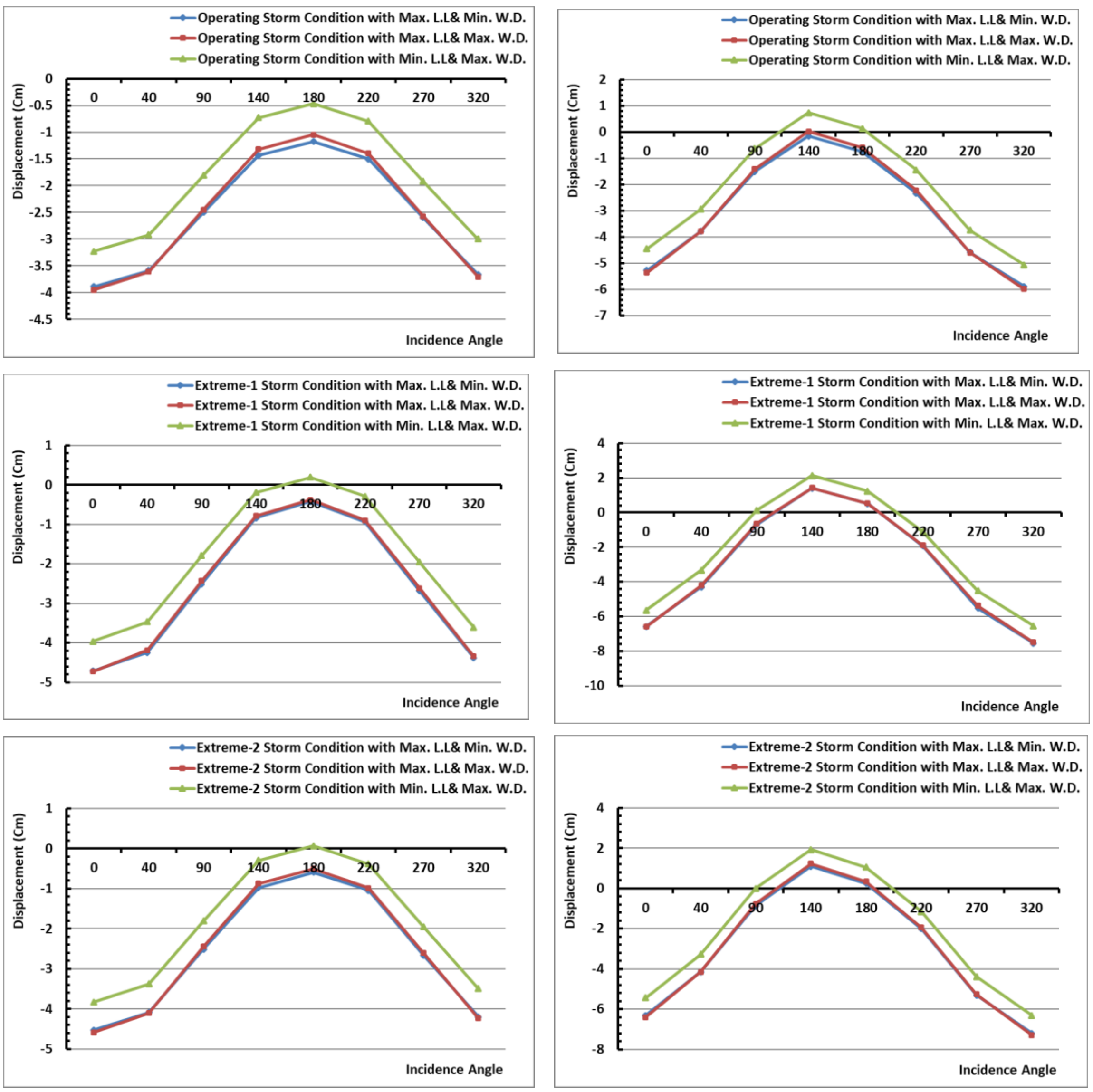
The vertical displacements (Z-direction) for the offshore platform legs according to the three storm conditions with respect to the incidence angles of environmental loads direction are illustrated in Figs. 11 and 12. The vertical displacements for maximum top level for all legs have the same behavior as the maximum value appear with angle environmental load direction 0 degree then decrease until 180 degree after that increase again. The water depth variations do not influence the vertical displacement, but the live loads have effect on displacement values while the values decrease accompanying with all storms which have minimum live load. All load cases produce negative vertical displacements for all top of legs according to variation of live loads and water depth with respect to all angles of environmental loads directions. Only some of positive vertical displacements appear with 180 degree at extremes conditions. On other hand the vertical displacements at mudline levels illustrated in right hand of Figs. 11 and 12.



(a) Top level (b) Mudline level

Fig. 11 Vertical displacement for top level (left) and mudline level (right) of (leg A-1) for three storm conditions according to the angles of the environmental loads' directions

Each leg has different configuration for vertical displacements at mudline level, but it has the same configuration for the different three storm conditions operating, extreme-1and extreme-2 storm conditions and values difference according to load cases in each storm condition are very low. For leg A-1, the vertical displacements change from positive to negative (tension to compression) displacement which the maximum positive value accompanying with environmental direction loads of 40 degree and the maximum negative accompanying with environmental direction loads of 220 degree. The two maximum values appear with perpendicular to jacket diagonal directions.



(a) Top level (b) Mudline level

Fig. 12 Vertical displacement for top level (left) and mudline level (right) of (leg A-2) for three storm conditions according to the angles of the environmental loads' directions

Fig. 12 (right hand side) displayed the vertical displacements for mudline level for leg A-2 which, have the same behavior as start negative then change to positive with angle 140 degree then change again to negative and the maximum value appear with angle environmental load direction 320 degree. The vertical deflections for the other two legs on row B of the platform at the mudline levels have an opposite behavior to that of legs on row A. A member check of a frame's structural member is performed to assess whether the member is subjected to acceptable stress levels. Joint unit checks: a punching shear check is carried out on the brace member at a joint to assess the shear through the chord. As for the other checks, these assessments are made using a punching shear interaction equation that delivers a usage factor. The displacements of the platform in existing condition are in allowable range. The allowable lateral displacement at the pile-head is taken to be 10% of the pile diameter as stated by ASTM STP-835. A similar ratio has been considered for the allowable axial displacement at the pile-head.

7. Conclusions

In-place analysis for offshore platforms is required to make proper design for new structures and true assessment for existing structures. In addition, ensure the structural integrity of platforms components under the maximum and minimum operating loads and environmental conditions. In-place analysis was carried out to verify the robustness and capability of structural members with all appurtenances to support the applied loads in either operating condition or storm conditions. This paper represents a case study of the existing fixed offshore platform located in Suez Gulf by in-place strength analysis. The objectives of this analysis are to verify whether the platform can meet the structural requirements, as per API RP 2A, for the In-place extreme met-ocean loading. The Gulf of Suez region is of high economic importance with promising future prospective for more offshore projects. A case study for a typical fixed platform located in the entrance of Gulf area is presented. The in-place performance of the offshore platform is assessed using a finite element method by structural analysis computer system (SACS). The in-place analysis performed for the studied platform under 72 different load combinations cases divided in three main storm conditions, called as operation storm, extreme storm-1, and extreme storm-2 conditions. The main factors which drive and control the different storm conditions are the environmental loads return periods and the water depth variation.

The results show that the studied platform has adequate strength and can resist environmental load. Analysis results show that the drift of platform is acceptable as it does not exceed the allowable drift limit. Each platform leg has different configuration for vertical displacements at mudline level, but it has the same configuration for the different three storm conditions operating, extreme-1 and extreme-2 storm conditions and values difference according to load cases in each storm condition are very low. The directions of environmental loads and water depth variations have an important effect on the results of the in-place analysis behavior. The live loads variations have a role in appearing of tension of the platform foundation. As a result, from the pile-soil interaction (PSI) analysis, 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 loads of the offshore platform well and real simulation of offshore foundation modeling in the in-place analysis. Likewise, the consideration of the topsides and support structure as one entity, the offshore structures, results in a potentially more appropriate and more economic designs. The result of the study shows that the in-place response investigation is quite crucial as well as environment for safe design and operation of offshore platform by consideration of the topsides and support structure as one entity. Also, as result from the study and in case of reassessment of platform to extension their life, the control of platform exposure to live load has an important role to maintain the platform responses against the variation probability of environmental loads.

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