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Structural performance assessment of fixed offshore platform based on 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. The structural integrity of platform components under the maximum and minimum operating loads of environmental conditions is required for risk assessment and inspection plan development. In-place analyses have been executed to check that the structural member with all appurtenances robustness and capability to support the applied loads in either storm condition or operating condition. 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 analysis includes interpretation of dynamic design parameters based on the available site-specific data, together with foundation design recommendations for in-place loading conditions. 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 important effects on the results of the in-place analysis behavior. The result shows that the in-place analysis is quite crucial for safe design and operation of offshore platform and assessment for existing offshore structures.

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

1. Introduction

The number of offshore platforms in the world is increasing year by year, most of which are of fixed jacket-type platforms located in 30 m to 200 m depth for oil and gas exploration and production. The analysis, design, and construction of offshore structures compatible with the extreme offshore environmental conditions is a most challenging task. Over the normal conditions met by land-based structures, offshore structures have added complication of being placed in an ocean environment where hydrodynamic interaction effects and dynamic response become major concerns in their design (Gudmestad 2000, Haritos 2007). Advancements in the oil and gas recovery

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from several areas have raised the concern for the structural integrity of platform elements under the maximum and minimum operating loads of environmental conditions that is essentially required for risk assessment and inspection plan development. In-place analysis for offshore platforms aims to control the global completeness of the platform against too early failure. Assessment of jacket platforms exposed to environmental loads larger than their original design loading frequently reveals that the capacity of the structural system is controlled 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 2015, Ishwarya *et al*. 2016). A sum of 337 failure modes have been recognized and analyzed by experts indicating nearly 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. Gebara *et al*. (2000) assessed the performance of the jacket platform under subsidence and performed ultimate strength and reliability analyses for four levels of sea floor subsidence. It was concluded that the wave load should be included to ensure that the structural integrity of the offshore platform meets the design and assessment requirements (Golafshani *et al*. 2009, Elsayed *et al*. 2015, 2016). The risk assessment uses the available platform's data to identify the platforms most at risk, hence defines the inspection interval and general inspection requirements. The quantitative method involves either structural analysis results with a dedicated metocean hazard or structural reliability method (Guédé 2019). The reliability of structures is affected by various impacts that generally have a negative effect, from extreme weather conditions, due to climate change to natural or man-made hazards. In recent years, extreme loading has had an enormous impact on the resilience of structures as one of the most important characteristics of the sound design of structures, besides the structural integrity and robustness (Ademovic and Ibrahimbegovic 2020). Due to currently frequent extreme events, the design philosophy is shifting from Performance-Based Design to Resilience-Based Design and from unit to system (community) resilience.

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

Offshore structures should be designed for severe environmental loads and strict requirements should be 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 that the structure natural frequency approaches the predominant wave frequency, the analysis must take care of response amplification at the wave period (Abdel Raheem 2013, Khandelwal 2018).

This paper represents a case study of a fixed offshore platform located in Gulf of Suez by in-place strength analysis. The modelling of offshore platform structure which includes the top side platform and the support structure is elaborated including aspects of structure modeling, piled structures, and hydrodynamic loading. The offshore structure model and environmental parameters for the site location of platform under consideration are developed. A three-dimensional finite element model is formulated to determine the stresses and displacements in a 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.

2. Platform description and modelling

2.1 Description of the platform

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. There are nine conductors and three risers connected by the platform. The top of air gap zone (wave-deck clearance) located at elevation (+) 6.52 m with respect to LAT. The platform consists of three parts as shown in Fig. 3. First, Topside, formed from 4 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 6 horizontal brace levels, 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

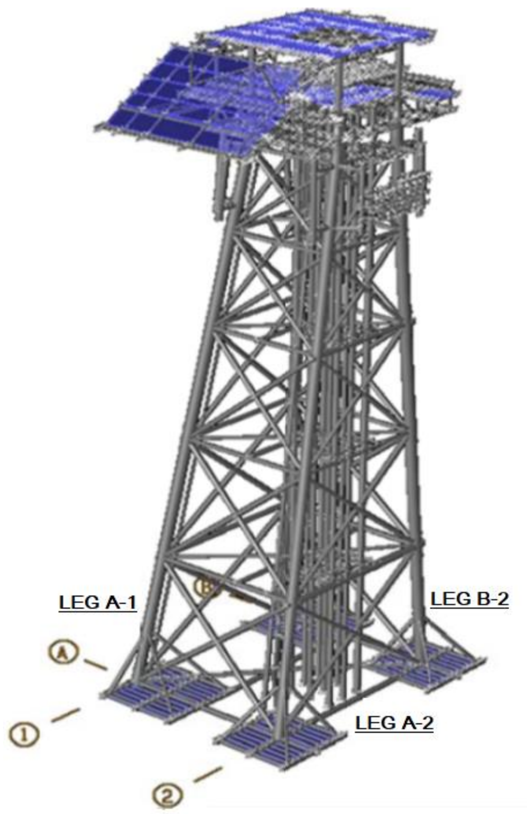
Fig. 2 Map Location of the offshore platform

Fig. 3 Photos of the study platform at site

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, Poisson's Ratio 0.3, Coefficient of Thermal Expansion 1.12 E-4/C^o and material yield strength is equal to 345 MPa for thickness ≤40 mm and 335 MPa for thickness >40mm.

2.2 Substructure and topside modelling

A three-dimensional finite element model of the substructure and topside is prepared reflecting its in-place condition. The structural model includes 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 is carried out using the SACS suite software (Bentley Systems 2011) which including jacket, deck, piles, stubs and supporting guides for conductors, risers and appurtenances is used for analysis. All members are modeled as 3D frame elements that are rigidly connected to each other. Shim plate centralizers inside the jacket leg at horizontal planes are 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 is simulated by modeling both pile and jacket members rigidly framing to those joints. All conical transitions are modelled to account for the stress concentration around the cone joints. Helideck plating is modeled as membrane element to simulate its participation in the overall lateral stability. Solar panels are 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 are modeled to calculate their weight and buoyancy by SACS program. All jacket appurtenances like boat landing, risers, mudmats, barge bumpers and conductors are 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 is eliminated. The coordinate system is the right hand cartesian system with the origin at the center of the deck legs and lies at LAT (Lowest Astronomical Tide) elevation, with (+)ve Z-axis



Fig, 4 3D finite element model of the substructure and topside

vertically upward and the (+)ve X-axis pointing to the platform east then the (+)ve Y-axis determined using the right hand rule. The four legs are defined by two pairs of horizontal axes 1 and 2 parallel to x-axis, A and B parallel to y-axis as shown in Fig. 4. The environmental loading on anodes is included by globally increasing the drag and inertia coefficients by 5% to 7%. The weight of anodes is inputted as joint loading at the appropriate nodes. In general, all the jacket miscellaneous and appurtenance structures whose are required to withstand the in-place loading conditions are accurately covered in the computer model with proper releases, such that their hydrodynamic and stiffness characteristics are truly represented.

2.3 Foundation modelling and simulation of pile to jacket interaction

The modelling of foundation piles and conductor piles is constructed based on the pile/conductor size and penetration as defined in the design shop 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 pile-soil interactions are modelled as a set of nonlinear springs in the form of soil lateral capacities (P-y), axial values (T-z) and end bearing values (Q-z) curves. The geotechnical properties of underlying soil are used to generate the pile axial adhesion, skin friction and bearing capacity based on API-RP2A recommendations (API 2014). Soil properties at the site are used to generate the pile lateral soil properties in the form of load deflection curves, based on API-RP2A recommendations. Group effect for the piles and conductors is introduced for piles with centre to centre spacing less than 8 times the diameter of the piles. As all the conductors are installed along with the jacket, conductors are 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 are checked to assure they don't exceed the conductors attracted hydrodynamic loads and according the conductors don't share in the resisting the shear load on the platform. Iterative analysis is 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 is carried out for the combination load as basic load cases. This is 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 are 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 is 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 is set to zero by giving the member dimension overrides. Piles and legs are considered flooded for in-place analysis.

2.4 Hydrodynamic modelling

A rough type marine growth is considered in the analysis as from elevation (+2 m) to (- 15 m) with respect to 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 is input as 1308 kg/m^3 rather than 1400 kg/m^3 in order not to consider a contingency over the marine growth weight. This approach is 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 are taken as C_d equal to 0.683 and C_m equal to 1.68 for smooth surface which for rough surface C_d and C_m equal to 1.103 and 1.26 respectively. Drag and inertia coefficients are 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 don't attract environmental loads or have no buoyancy. Members are modeled as flooded or non-flooded as its position. General coupling strategies for multi-physics coupling and interaction problems for fluid-structure interaction that allows to reuse existing code are proposed (Ibrahimbegovic *et al*. 2016, Abdel Raheem *et al*. 2018). An implicit partitioned algorithm separating fluid from structural iterations is developed for the solution of nonlinear fluid-structure interaction, where different space discretization is required using finite elements for structure and finite volume for fluid (Kassiotis *et al*. 2011 a, b, Hajdo *et al*. 2020). Data transfer at fluid-structure is simplified by leading to equivalent follower pressure load (Boujelben *et al*. 2020).

Table 1 Dry weights of the modeled items and Marine Growth

No.	Item	Net Weight (tone)
1	Modeled Deck Structure	178
2	Modeled Jacket Structure	656
3	Above Mudline Piles	505
4	Appurtenances	
	• 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	0.20kN/m^2	108.72	
2	Steel Grating	0.50 KN/m^2	60.17	
3	10 mm Plating	0.77 KN/m^2	232.95	Excluding Mudmat plating
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

2.5 Structural loading

The individual basic 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.

2.5.1 Self weight

The self-weight of all structural members of the jacket model is 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 given in Table 1. Weight of un-modeled items like anodes, grating, handrail, etc. are 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 2.

2.5.2 Buoyancy

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 is calculated and explicitly applied as loads on the members. Remaining jacket tubular members is considered buoyant. Buoyancy acting on un-modeled items below MSL is 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

Table 3 live loads used for the different design cases considered in the analysis, UDL (KN/m^2)

Area	Flooring & Stringers	Main Deck Girders	Main Truss Framing	Substructure
Laydown and Storage Areas	20	15	10	5*
Stairways, Access Platforms and Walkways	5	2.5	2.5	-
Helideck	25	15	10	3*
Open Areas	5	5	5	2.5

* For cases, where the gravity loads involve minimum gravity loads (maximum tension on piles), these loads are eliminated.

Table 4 Total blanket live loads considered in the analysis

NO.	Item	Weight (T)
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

into combinations the dead weight without buoyancy is used to represent the weight contingencies on self-weight only.

2.5.3 Live and equipment loads

Live loads are 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 are 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 are obtained from the weight control report (Gross weights rather than net weights are used to enable applying separate contingency for each equipment) and input as joint and/or member loads.

2.5.4 Environmental loading

Provisions and requirements of the American Petroleum Institute (API 1993, 2010, 2014) and the project basis of design are introduced (Abdel Raheem *et al*. 2020a, b). Six environmental loading conditions are introduced in Table 5. Wind, wave and current are assumed to act concurrently in the same direction. Eight loading directions are 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 $T_{H\max}$) are taken from the metaocean criteria. Doppler Effect of the current on wave is accounted for by calculating the apparent period for all the considered waves. SEA STATE program calculates the apparent period based on the actual wave period, water depth and current velocity. Two-dimensional wave kinematic are determined from the stream wave theory for the specified wave height, water depth, and apparent period. The stream function order is automatically determined by SEASTATE. Wave kinematics factor is taken equal to 0.866. A series of wave stepping runs are carried through

Table 5 Environmental loading conditions considered in the analysis

Condition	Return Period (Year)			Water Depth (w.r.t L.A.T) 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 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 are taken from the metaocean criteria for offshore platform position. Profiles are nonlinearly stretched up to wave crests. Current blockage factors are 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 is 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 are extracted from the metaocean criteria and used for analysis of the substructure (jacket structure). Omni directional 1-minute mean wind speeds are extracted from the metaocean criteria and used for analysis of the top structure (deck structure). Flat wind areas are generated for wind loads imposed on equipment/bulks installed on the deck levels.

2.5.5 Design wave and wind loads

Orthogonal and diagonal wave directions are analyzed for the in-place condition. The Morison equation is used for converting the velocity and acceleration terms into resultant forces and is extended to consider arbitrary orientations of the structural members. 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 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.

3. Methodology and numerical analysis

The procedure for reassessment of offshore platform for this study is refers to the standard AISC-ASD and API RP2A-WSD (AISC 2005, API 2014). In-place analysis is performed using structure analysis computer program by considering all loads conditions for Still Water Case, 1-Year Condition and 100-Year Condition. Still Water condition cases combines maximum load operation 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. Design and strength of structures are expressed

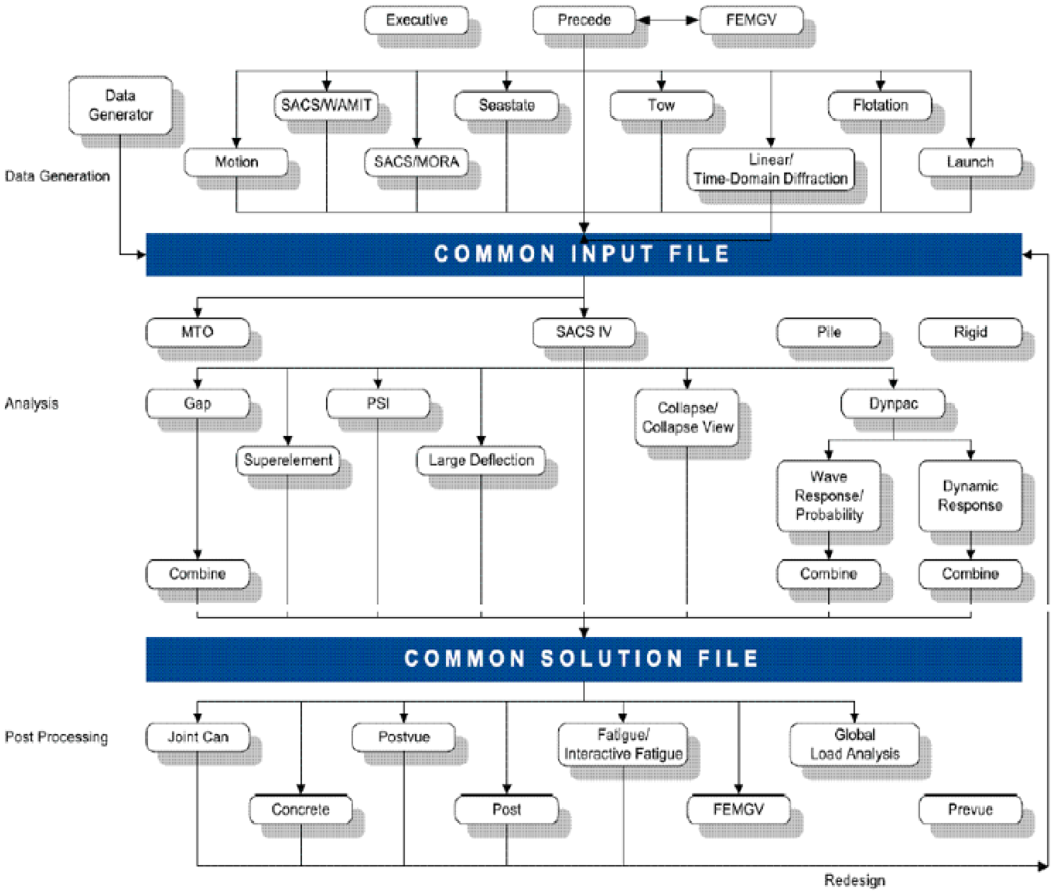


Fig. 5 Modelling, analysis procedure for in-place behavior of the platform based on SACS program

in Unity Checks (UC) 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 to meet the requirement as stipulated in international standards (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 condition or extreme condition. The components of the platforms 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 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 (e.g., 100-Year Return Period) conditions must meet the standard requirements for the design and reassessment of fixed offshore structures (Henry *et al*. 2017). Fig. 5 introduces the Modelling, analysis procedure for in-place behavior of the platform based on SACS program (Bentley Systems. 2011).

3.1 Vibration characteristics of the platform

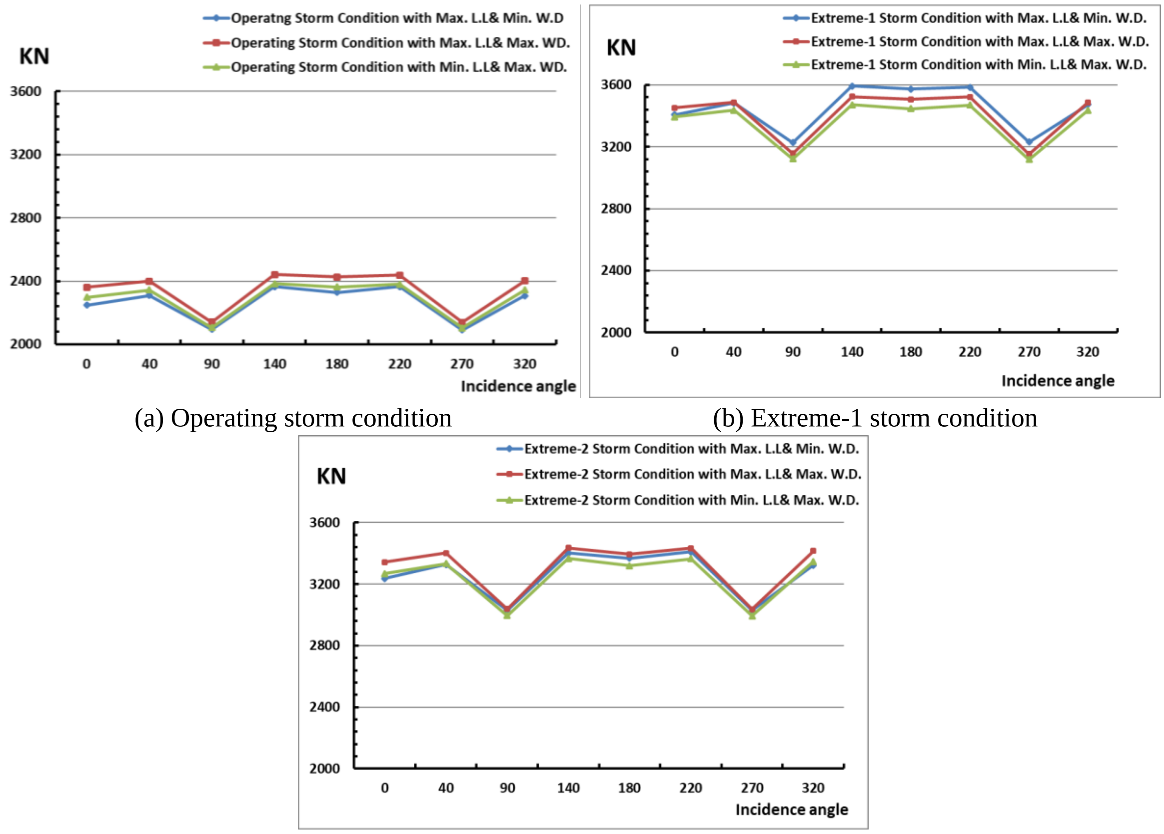
To acquire the dynamic characteristic of the platform, a modal analysis is performed using the DYNPAC module of the SACS package. It uses a set of master (retained) degrees of freedom, selected to cover intersection joints, to extract the Eigen values (periods) and Eigen vectors (mode shapes). All stiffness 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. The first 40 vibration mode shapes are extracted to properly simulate the dynamic response of the platform. Mass is 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 is then carried out to generate the dynamic characteristics of the platform including vibration mode shapes and natural periods. The first three mode shapes are the dominant mode shapes correspond to sway, surge, and torsion modes of the platform. First 9 frequencies and natural periods, based on the platform data and site foundation characteristic, are calculated and are show in Table 6. Mode shapes represent the shape that the platform will vibrate in free motion and the same shape dominates the motion of the platform during environmental excitation. The first mode of vibration is the one of primary interest as the first mode has the largest contribution to the platform motion during environmental excitation.

3.2 In-place analysis for three different storm conditions

The in-place analysis for the studied platform is carried out under 72 different load combinations that are divided in three main storm conditions: 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 in the study focus on main subjects as base shear, overturning moment, and joints displacement. The details of each subject will illustrate and display in the following paragraphs to display the information that help in the assessment of the structure under in-place analysis due to the different combinations of the

Table 6 Dynamic characteristic of the offshore platform case study

MODE	FREQ. (CPS, Hz)	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



(a) Operating storm condition (b) Extreme-1 storm condition

(c) Extreme-2 storm condition

Fig. 6 Total applied horizontal loads for different incidence angles of the environmental loads' direction

participation of the environmental storm conditions and gravity loads.

3.2.1 Base shear and overturning moment responses

Total applied horizontal loads resultant that affect platform due to all load cases in different storm conditions are used in the analysis. Fig. 6 illustrated horizontal loads resultant in the operating storm condition due to two variables: live load and water depth with respect to the 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 as shown in the figure. The highest horizontal loads values achieved by the operating storm condition with maximum live load and maximum water depth. The Fig. 6 (b, C) showed the applied horizontal loads resultant in extreme-1 storm condition and extreme-2 storm condition, respectively. The two extreme storm conditions are behaved like the operating storm condition with respect to different 8 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 case 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 difference of values in the three environmental storm conditions as depicted in Fig. 6, then influence on the results of all analysis including straining actions, displacement, velocity and acceleration. The three environmental storm conditions follow the same configuration while, its values differ in small amount as

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^o
Extreme Storm-1	3592.07	221142	Minimum	140^o
Extreme Storm-2	3435.42	209108	Maximum	140^o

configured in figures. Only live loads have an important role in vertical loading which the minimum applied vertical loads accompanying with minimum live load and maximum water depth for all three environmental storm conditions. The water depth variations have not influence on the changing values of the vertical applied loads on the offshore platforms.

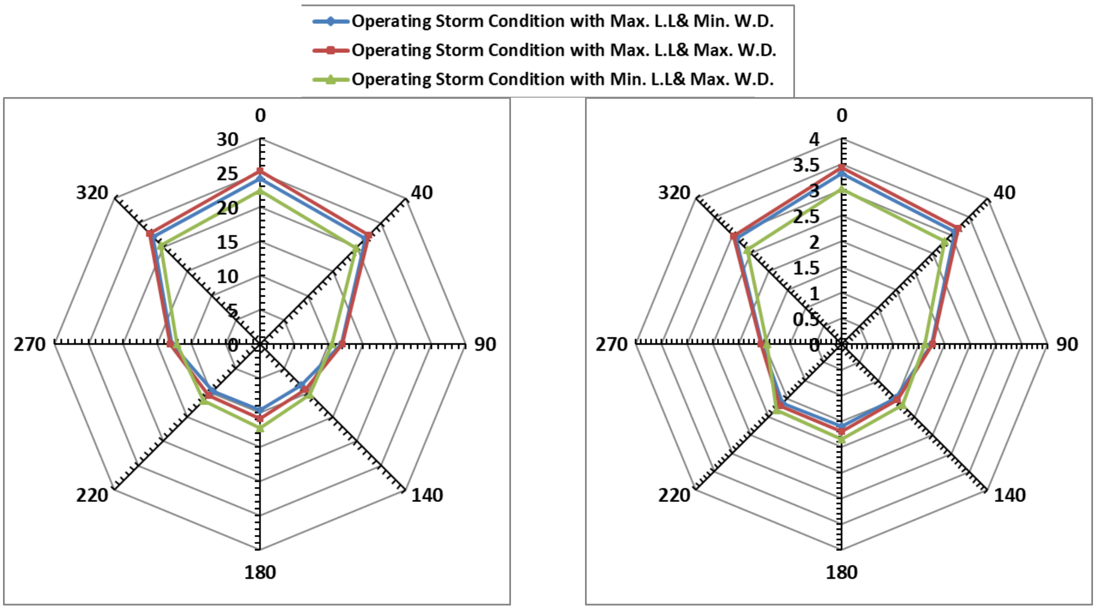
Some of the important checks in design and analysis of offshore platform are base shear and overturning moment which the platform as a whole act and behave as a cantilever supported on seabed and extended through sea water until designed height. Table 7 summarize the maximum base shear and overturning moment acting on the platform due to environmental loading cases in different storm conditions. The in-place structural analysis of the jacket structure is meant to determine 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°. For these wave directions the exposed surface area of the jacket is larger than any other directions and they 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.

3.2.2 Joints displacement response

The joints displacement are very important results from platform analysis due to all risers, pipelines, static/rotating equipment, instruments, and all control devices connect and fix to platform. The values of joints displacement of the platform will influence all things that connected to platform and the increasing of displacement more than the limits not only cause deformation and harmful for platform structure but also for all connections items and devices which could lead to hazards and disasters for all area.

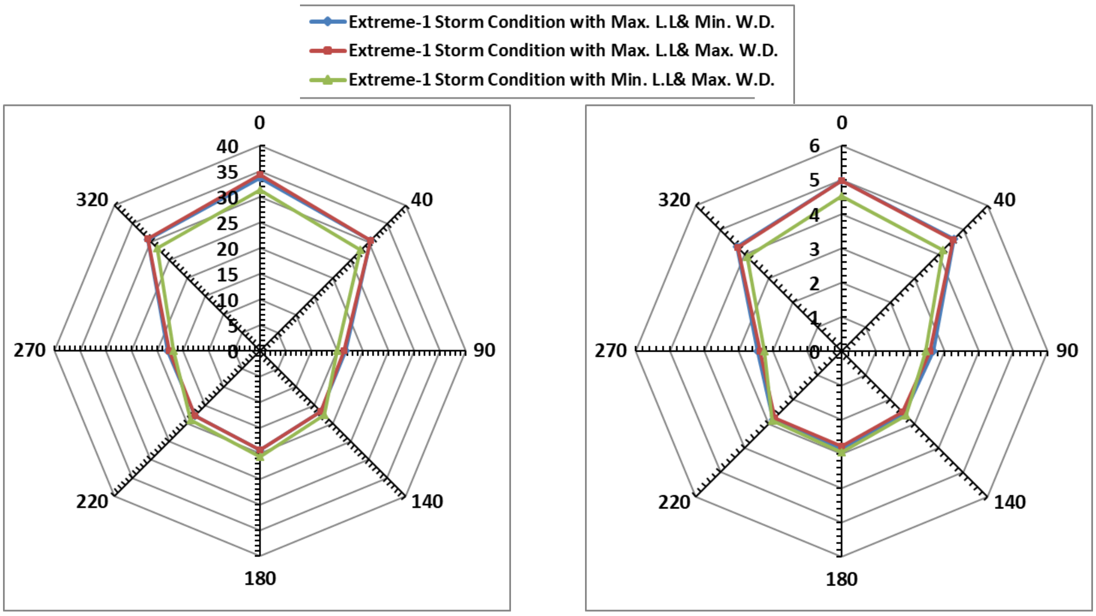
3.2.3 Horizontal displacement response

Horizontal displacements for offshore platform are considered one of the main important results from the analysis and have strong relations with the environmental loads. Figs. 7 -9 illustrated the absolute horizontal displacements for most top levels and mudline levels of the four legs for offshore platform case study under the three storm conditions (operating, extreme-1 and extreme-2 storm conditions) according to the 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



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

Fig. 7 Absolute horizontal displacement for operating storm condition for different incidence angles



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

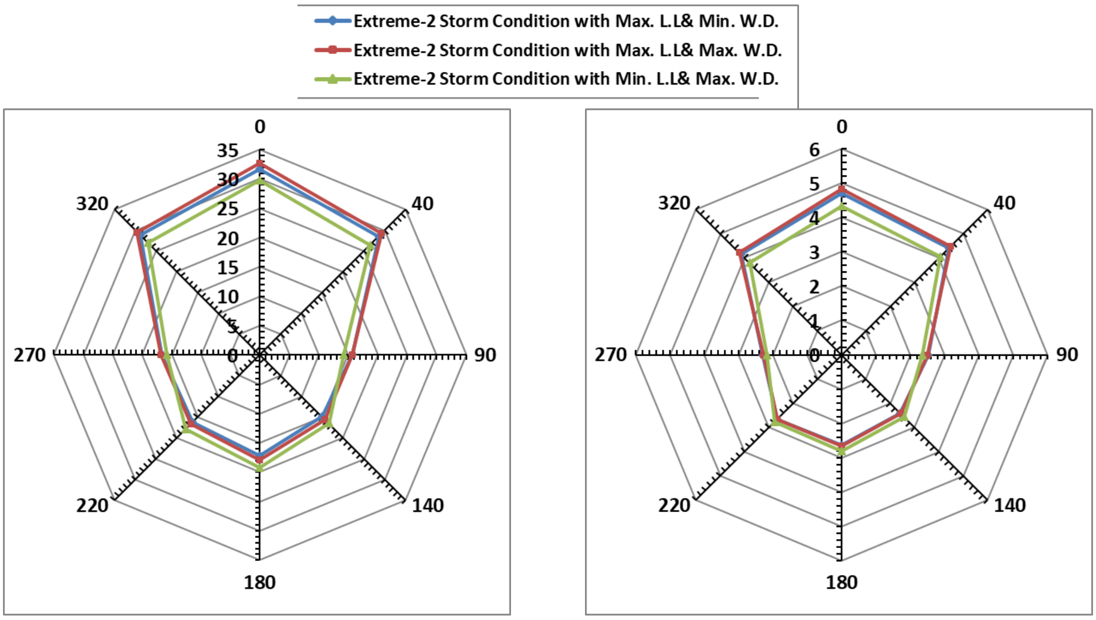
Fig. 8 Absolute horizontal displacement for extreme-1 storm condition for different incidence angles

live loads with maximum water depth).

The maximum live loads cases achieved the maximum displacement values in all environmental directions except environmental loads directions 140, 180 and 220 degree in all platform legs are be maximum with minimum live loads case in all storm conditions. For the operating storm conditions, the absolute horizontal displacements affected by variation of water depth in constant live loads with respect to environmental loads direction. 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 among 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). 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. All drift is acceptable as the allowable drift for platform equal to height/200=49.05.

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

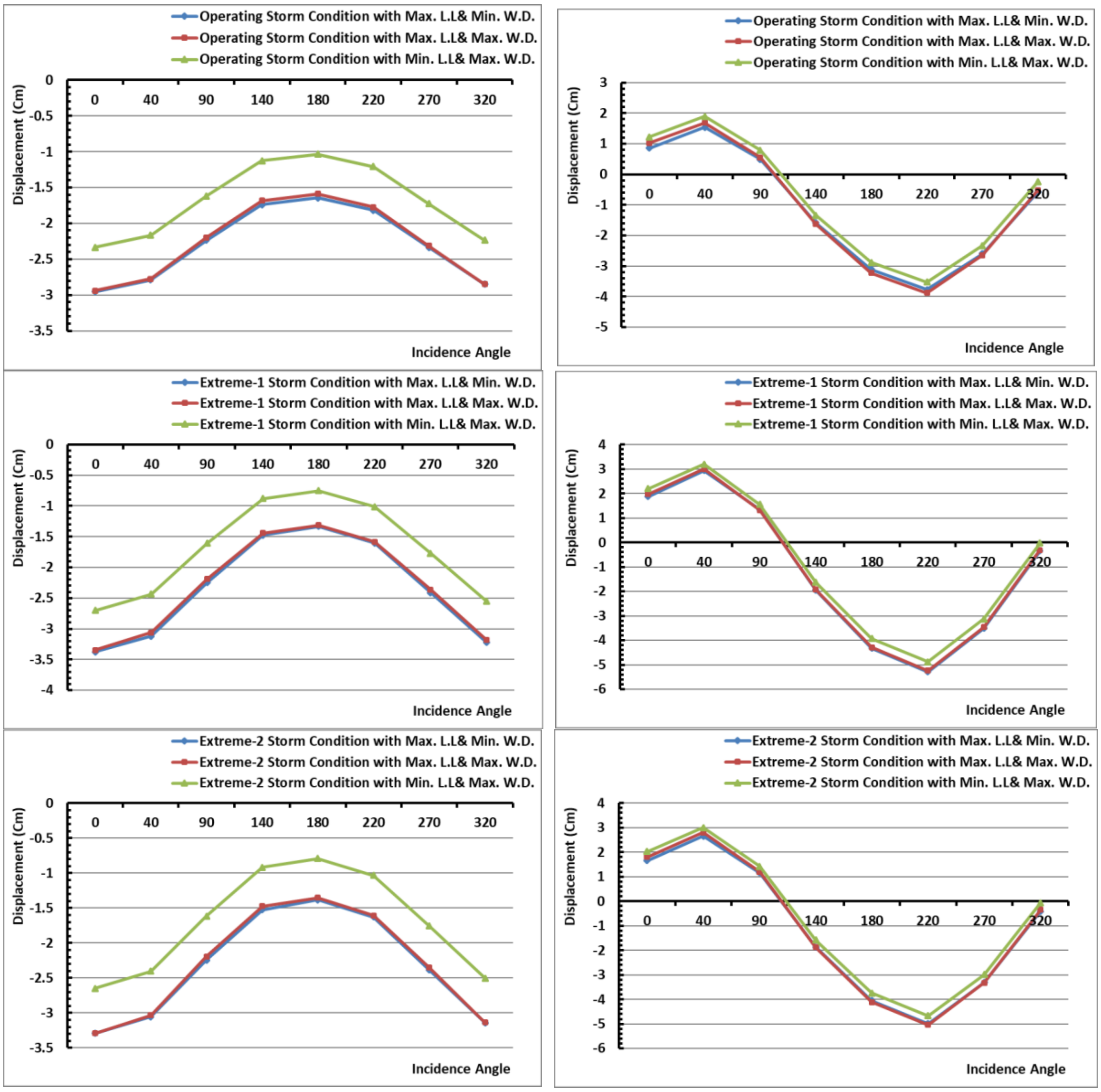
Leg	Levels	Maximum Absolute Values (cm)	Storm Condition	Relative Value (Drift) (cm)
A-1	Mudline (-78m)	4.97	Extreme-1	29.28
	Most Top (+20.1m)	34.25		
A-2	Mudline (-78m)	4.96	Extreme-1	29.24
	Most Top (+20.1m)	34.20		
B-1	Mudline (-78m)	4.88	Extreme-1	29.30
	Most Top (+20.1m)	34.18		
B-2	Mudline (-78m)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



LEG A-1 Top Level

Leg A-1 mudline level

Fig. 9 Absolute horizontal displacement for extreme-2 storm condition for different incidence angles

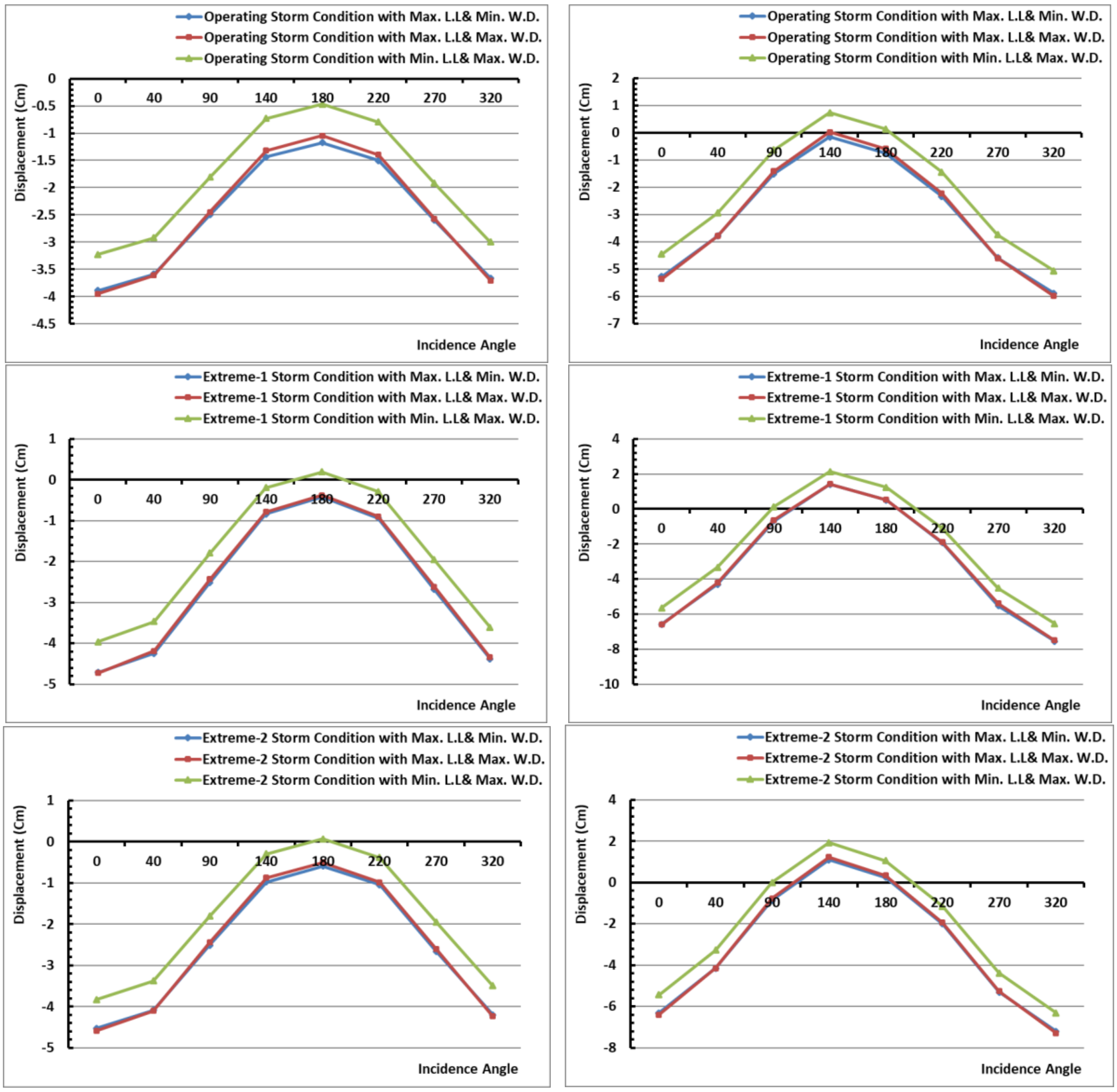


(a) Top level (b) Mudline level

Fig. 10 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

3.2.4 Vertical displacement response

The vertical displacements (Z-direction) for the offshore platform legs according to the three storm conditions with respect to the angles of environmental loads direction are illustrated in Figs. 10-11. 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° 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



(a) Top level (b) Mudline level

Fig. 11 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

storms which have minimum live load. All load cases produced 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. 10-11. 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

Table 9 Maximum unity check for members due to all load cases in deferent storm conditions

Item	Maximum Unity Check	
	Location	UC
Above Mudline Piles	Pile A-1	0.232
	Pile A-2	0.331
	Pile B-1	0.232
	Pile B-2	0.323
Jacket Legs	Leg A-1	0.580
	Leg A-2	0.622
	Leg B-1	0.560
	Leg B-2	0.632
Jacket Vertical Braces	Row A	0.283
	Row B	0.271
	Row 1	0.242
	Row 2	0.242
Jacket Horizontal Braces	Elevation - 77.985 m	0.662
	Elevation - 57.000 m	0.481
	Elevation - 38.000 m	0334
	Elevation - 21.000 m	0.192
	Elevation - 07.000 m	0.334
	Elevation + 05.000 m	0.462
Deck Legs	Leg A-1	0.522
	Leg A-2	0.471
	Leg B-1	0.512
	Leg B-2	0.471
Deck Braces	Row A	0.862
	Row B	0.891
	Row 1	0.723
	Row 2	0.641
Deck Truss Beams	Below Production Deck	0.641
	Production Deck	0.501
	Helideck	0.462
Deck Girders	Below Production Deck	0.290
	Production Deck	0.720
	Helideck	0.850
Deck Floor Beams	Below Production Deck	0.571
	Production Deck	0.500
	Helideck	0.732

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. Fig. 10 (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° then change again to negative and the maximum value appear with angle

Table 10 Maximum load unity check for joints due to all load cases

Item	Maximum Load Unity Check	
	Location	UC
Jacket Joints	Horizontal Brace - Horizontal Brace @ EL. (-)21.00 m	0.39
Deck Joints	Deck Truss Chord -Deck Braces Row B @ EL. (+) 18.50 m	0.57

Table 11 Maximum strength unity check for joints due to all load cases

Item	Maximum Strength Unity Check	
	Location	UC
Jacket Joints	Vertical Brace - Vertical Brace at Row-2	0.98
Deck Joints	Deck Truss Chord - Deck Leg A-2 at EL. (+) 18.50 m	0.60

environmental load direction 320 degree. The vertical deflections for mudline levels of the other two legs on row B of the platform have an opposite behavior to the two legs on row A which legs B-1 and B-2 opposite legs A-2 and A-1 respectively.

3.2.5 Member and joint unity check

The strength unity checks are for the primary joints which are required to fulfill the minimum capacity requirements by API RP 2A. All members have been checked against the requirements of API RP 2A. The analysis displayed that no failure was reported in any of the platform members, a summary for the maximum unity checks (UC) values due to all load cases in different storm conditions for various structural components were tabulated, Table 9. Hydrostatic collapse checks are included within the member unity check ratios.

Punching shear for the joints was checked against the requirements of API RP 2A by JOINTCAN module and were found adequate. The joint (as *X*, *T*, *K*, *Y* joint or combination of these) was classified by JOINTCAN based on the load path for each loading condition by the same program. The maximum load unity check (UC) values for both jacket and deck joints due to all load cases in different storm conditions for both jacket and deck joints are tabulated in Table 10. On other hand the maximum strength unity check (UC) values were displayed in Table 11.

4. Conclusions

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 studying platform which subjected to 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. All members have been checked against the requirements of API RP 2A and punching shear for the joints was checked also and were found adequate. Analysis results displayed that the drift of platform is acceptable as it is not increase than the allowable drift.

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 in the results of the in-place analysis behavior. The live loads variations have a role in appearing of tension of the platform foundation. 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.

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