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Influence of near-fault effects and of incident angle of earthquake waves on the seismic inelastic demands of a typical Jack-Up platform

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Abstract

In this paper, the potential influence of near-fault effects and of the incident angle of earthquake waves to the seismic response of a typical jack-up offshore platform is assessed by means of incremental dynamic analysis involving a three dimensional distributed plasticity finite element model. Two horizontal orthogonal strong ground motion components of a judicially chosen near-fault seismic record is considered to represent the input seismic action along different incident angles. The fault-normal component exhibits a prominent forward-directivity velocity pulse (pulse-like) whose period lies close to the fundamental natural period of the considered structure following a "worst case scenario" approach, while the fault-parallel component does not include such a pulse. Pertinent numerical data demonstrate that the fault normal component poses much higher seismic demands to the "prototype" jack-up structure considered compared to the fault parallel component. Further, significant variation in the collapse resistance/capacity values is observed among different incident angles especially for the "critical" fault normal component. It is concluded that the combined effect of forward-directivity phenomena and the orientation of deployed jack-up platforms with respect to neighbouring active seismic faults needs to be explicitly accounted for in site-specific seismic risk assessment studies. Further research is warranted to propose recommendations on optimum orientation of jack-up structures operating in the proximity of active seismic faults to minimize seismic risk.

Keywords: Near-fault seismic ground motions, forward directivity pulses, incremental dynamic analysis, seismic incident angle.

1. Introduction

The jack-up platform is a mobile/re-deployable steel offshore structure used world-wide in various oil and natural gas exploration and extraction activities and, more recently, to facilitate offshore wind energy harvesting. The deck of jack-up platforms is allowed to slide along steel space trusses (legs) which are supported by relatively shallow "pin-headed" foundations (the "spudcans") as shown in Figure 1 (e.g. [1,2]). The achieved penetration depth of spudcans and, consequently, the fixity level of the foundations depend heavily on local site soil conditions. In this regard, the dynamic properties of the lateral load resisting structural system of any particular jack-up platform vary significantly under the effect of many parameters, such as the deck height, the deck mass and its distribution, the site soil properties, and the spudcan penetration depth. Further, each jack-up platform is typically required to operate safely during its lifetime at several different sites and water depths under harsh environmental conditions involving lateral dynamic loads induced by the action of sea waves, sea currents and winds (e.g. [3]) . Thus, from a structural dynamics viewpoint, the design of a jack-up platform becomes a challenging task as design loads and operational conditions are quite uncertain.

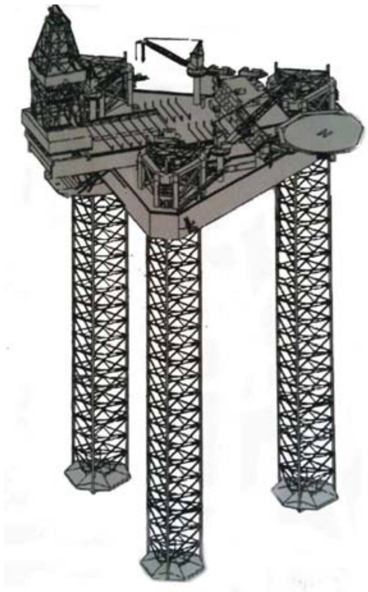

Figure 1. A typical three-legged Jack-Up platform (adopted from [1])

To this end, through design, monitoring, and operational experience accumulated over the years, robust three-legged jack-up platform designs have been evolved and can be taken as the "norm" of a typical offshore jack-up structure (e.g. [1, 4-6]). Further, rigorous structural integrity assessment of existing jack-ups against site-specific wind, wave, and current conditions is undertaken in accordance with pertinent guidelines (e.g. [7]). Additionally, when operating in active seismic regions, the potential action of earthquake induced strong ground motions, imparting inertial dynamic loads to any "fixed-based" off-shore platform including typical jack-up structures, need to be considered for design or for risk assessment purposes (e.g. [6, 8-10]).

In this regard, it is noted that, under certain seismological conditions, structures located within about 20km away from active seismic faults may be affected by earthquake induced ground motions (GMs) characterized by "forward-directivity" high amplitude long period pulses (e.g. [11-14]). Such low frequency pulses are a consequence of rapture directivity effects (e.g. [15]). Specifically, when the seismic fault rupture speed is similar to the propagation speed of the seismic shear wave front (generated by the fault rupture) towards a considered site, it is likely that the GM recorded at the site along the normal (perpendicular) direction to the seismic fault exhibit pulse-like (forward directivity) characteristics (see also Figure 2(a)). Statistical signal analysis of databanks of recorded GMs suggest that typical "pulse-like ground motions" (PLGMs) carry a significant fraction of the total energy of the recorded GM signal traces (e.g. [11,12]). To further illustrate this point, the acceleration trace of a typical PLGM is plotted in Figure 2(b) along with its energy distribution on the time-frequency plane (Figure 2(d)). The latter has been obtained by processing the considered PLGM via the harmonic wavelet transform and plotting the squared magnitude of the obtained harmonic wavelet coefficients versus (central) time and (central) frequency [16-18]. Similar plots pertaining to a typical far field (pulse-free) GM are juxtaposed in Figures 2(c) and 2€. Evidently, most of the energy of the PLGM (warm colors in Figures 2(d)) is well concentrated in time at relatively low frequencies, which indicate the existence of a low frequency/long period pulse. On the antipode, the energy of the far-field GM is well scattered in time and spreads across significantly higher frequencies.

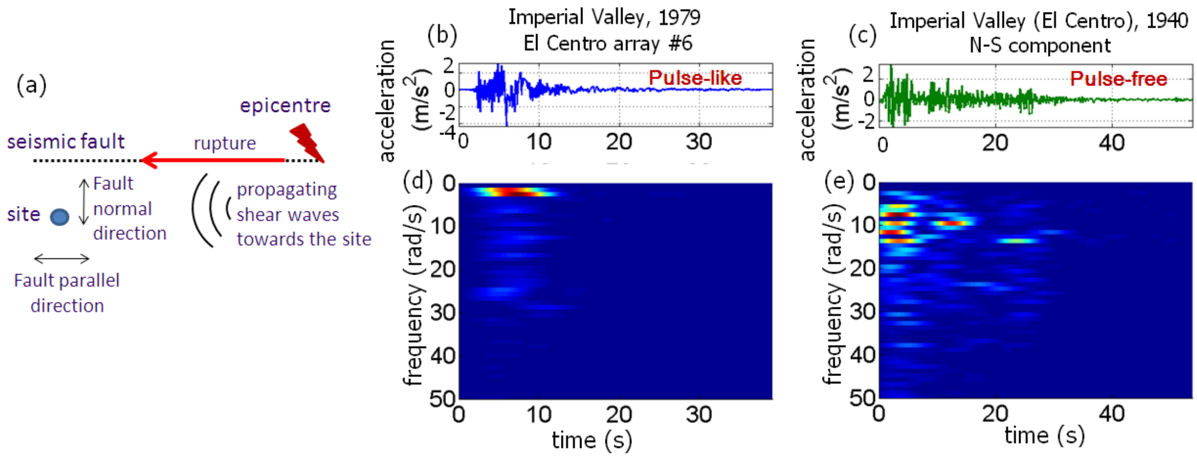

Figure 2. (a) Generation of forward directivity effects, (b),(c) Acceleration trace of a typical pulse-like and pulse-free ground motion, respectively, (d),(e) Harmonic wavelet based time-frequency representation of a typical pulse-like and pulse-free ground motion, respectively.

Consequently, compared with typical far-field GMs, PLGMs impose significantly higher ductility and strength demands to relatively flexible structures, such as jack-up platforms, with periods close to the dominant period of the pulse(s) T_p. In this respect, it has been recently recognized by the earthquake engineering community that accounting for forward directivity effects in the seismic design of new relatively flexible structures and in assessing the seismic vulnerability of the existing ones in near-fault environments is an important consideration (e.g. [13,19]). Indeed, there exist tenths of recorded GMs associated with various historical seismic events from which pulses with T_p lying in the range of 2s to 10s have been identified (e.g. [12]). Noticeably, this is the range that the fundamental period of jack-up platforms falls within during normal operations (e.g. [1,4-6]). Nevertheless, assessing the influence of forward directivity effects for typical jack-up platforms has not been explicitly considered in the open literature.

To this end, this paper considers a prototype three dimensional (3D) finite element (FE) model of a typical three-legged jack-up offshore platform subjected to a judicially chosen horizontal orthogonal pair of near-fault GMs. Adopting a “worst case scenario” approach, the fault-normal component contains a pulse whose period is close to the fundamental period of the considered FE model. Seismic performance assessment is accomplished by means of incremental dynamic analysis (IDA): a recent structural analysis approach [20,21] which has been extensively used for the seismic assessment of a wide variety of structures (e.g. [13,19,22-23]), including steel jack-up offshore platforms [24] within a performance-based design context. IDA involves a series of non-linear response history analyses (RHA) to be performed considering appropriately selected and scaled GMs corresponding to site specific earthquake scenarios. Since no provision is included in pertinent seismic codes of practice (e.g. [25]) with regards to the orientation/incidence angle of the seismic waves (accelerograms) in performing non-linear RHA, the considered pair of GMs is applied to the jack-up platform at different in-plan angles to further assess the influence of earthquake directionality to the seismic response demands of jack-up structures (see e.g. [26]).

The remainder of the paper is organized as follows. Section 2 provides a brief qualitative introduction to IDA and explains certain details related to its application for assessing jack-up platforms for the purposes of this study. Section 3 describes to some detail the adopted 3D prototype FE model of a typical jack-up platform. Section 4 reports numerical results on the influence of forward directivity effects and of the incident angle of the input seismic action on the seismic performance of the considered jack-up platform. Finally, section 5 summarizes concluding remarks.

2. Incremental dynamic analysis (IDA) and its application for jack-up platforms

Incremental Dynamic Analysis (IDA) is a structural analysis procedure aiming to derive a “one to one” mapping of different levels of the input seismic action onto judicially chosen peak structural response quantities obtained from non-linear response history analyses (RHA) [20,21]. The “level” or “intensity” of the seismic input action is expressed by means of a single scalar intensity measure (IM) or of a collection (vector) of IMs. Commonly used IMs include the peak ground acceleration (PGA) and the (pseudo-) spectral acceleration at the fundamental natural period of the structure under analysis $S_a(T_1)$. The IM levels are defined to force a structure all the way from elastic response to final global dynamic instability (collapse). Further, the peak (inelastic) seismic demand is expressed by means of an “engineering demand parameter” (EDP), such as the peak lateral displacement measured at a certain point on the structure along the direction of the seismic action. IDA can readily account for the inherent (aleatoric) uncertainty of the earthquake induced ground motion (GM) by considering a collection of recorded GMs corresponding to specific earthquake scenarios (e.g. moment magnitude, epicentral distance etc.) as input to perform RHAs for various IMs. IDA data results for each GM considered in the analysis are commonly represented in the form of IM vs EDP graphs (IDA curves). In Figure 2 a typical IDA curve corresponding to a single GM is shown. Each “dot” is derived from a RHA and the IDA curve is constructed via (spline) interpolation [20,21]. In most of the cases, the non-linear response region can be readily identified, while a “stiffening” pattern (EDP reduces from an increased IM as shown in Figure 2) and other complex non-linear phenomena may reveal themselves depending on the structure and the properties of the considered GM for specific IM and EDP measures. In this regard, IDA can be viewed as a “dynamic version” of the well-known static inelastic (pushover) analysis widely used by the marine engineering and earthquake engineering communities for structural design and assessment purposes against (lateral) dynamic loads.

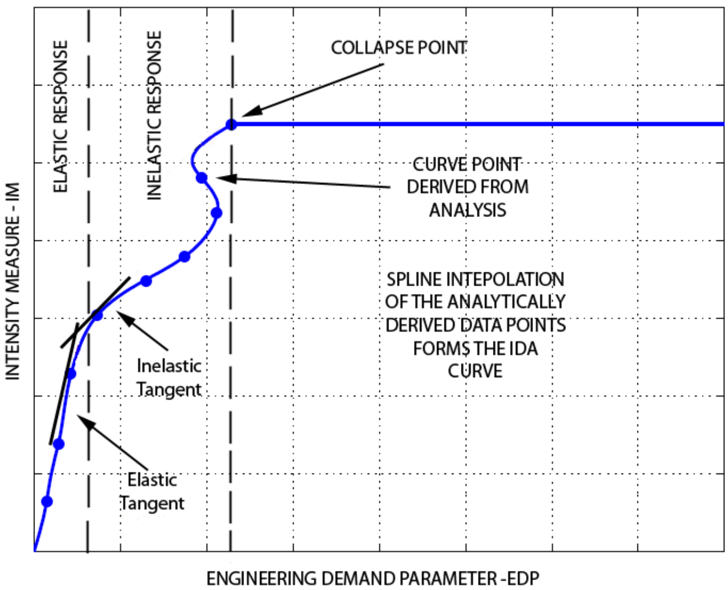

Figure 3. Typical IDA curve with possible key patterns/characteristics

From a computational viewpoint, IDA is quite intensive and several algorithms have been proposed in the literature to improve its efficiency (see e.g. [27]). For the purposes of this study, the "HCOUPER" high performance computing unit housed in the Civil Engineering Department of Aristotle University, Thessaloniki, Greece (http://hcouper.weebly.com/english.html) has been used. A series "hunt and fill" algorithm combined with a pseudo-parallelization scheme that makes use of the multithread capabilities of the available computer was developed in MATLAB® [28] and used to derive all the ensuing numerical results.

A further major concern in performing IDA is the proper selection of the IM and the EDP quantities which is case-specific and an issue open to research. Detailed discussions on the current state of knowledge regarding the selection of IMs to "scale" near fault pulse-like GMs can be found in [13,19,29,30]. Recently, Sehhati et al. [13] have demonstrated that the peak ground velocity (PGV) is a better qualified IM compared to the commonly used PGA and $S_a(T_1)$ for the seismic assessment of steel multi-storey moment-resisting framed structures within the performance-based seismic design approach by considering ensembles of pulse-free and pulse-like GMs with various pulse periods. Moreover, the maximum inter-storey drift ratio (relative floor displacement over storey height) is known to be well-correlated with elastic and inelastic structural demand and accumulated damage for steel buildings (e.g. [31]) and is the most commonly used EDP for such structures (e.g. [13,20,21]). To this end, given the focus of this work on pulse-like GMs and the nature of steel jack-up structures, the PGV is adopted as the IM of choice in the numerical work reported in section 4. Moreover, the geometry of typical steel truss jack-up legs allows for defining the maximum observed drift for each steel truss "storey" from all jack-up legs along the direction of the seismic action as shown in Figure 3.

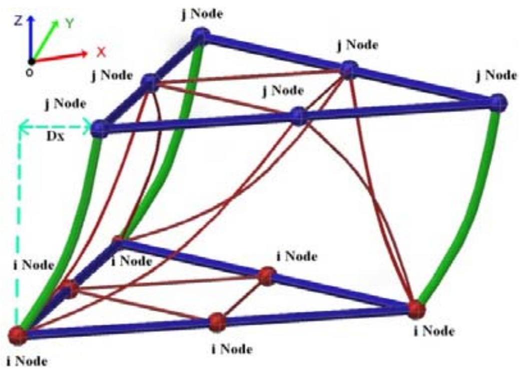

Figure 4. Illustration of EDP considered: inter-storey drift along axis of seismic action OX (Dx/"floor height") [24].

3. Adopted jack-up finite element model

3.1 Geometry and modelling assumptions

IDA is usually applied to two-dimensional (2D) finite element (FE) models to study the effects of the strong ground motion along the "principal" axes of structures. Nevertheless, three-dimensional (3D) FE models need to be considered in performing IDA to capture the inelastic behaviour of irregular in plan structures accounting for their rotational motion (e.g. [31]), as in the case of off-shore platforms (e.g. [22,24]), or to account for the impact of the earthquake incident angle (e.g. [26]). In this regard, a "prototype" 3D FE model of a typical jack-up platform is developed in the OpenSEES FE simulation platform for earthquake engineering [32] which achieves a reasonable compromise between detail in modelling and computational cost in conducting IDA (see also [24]).

Details on the geometry of the considered model in elevation and in plan are shown in Figures 4 and 5(a), respectively. The model is based on a structure considered by Gjerde et al. [4] (see also [24]).

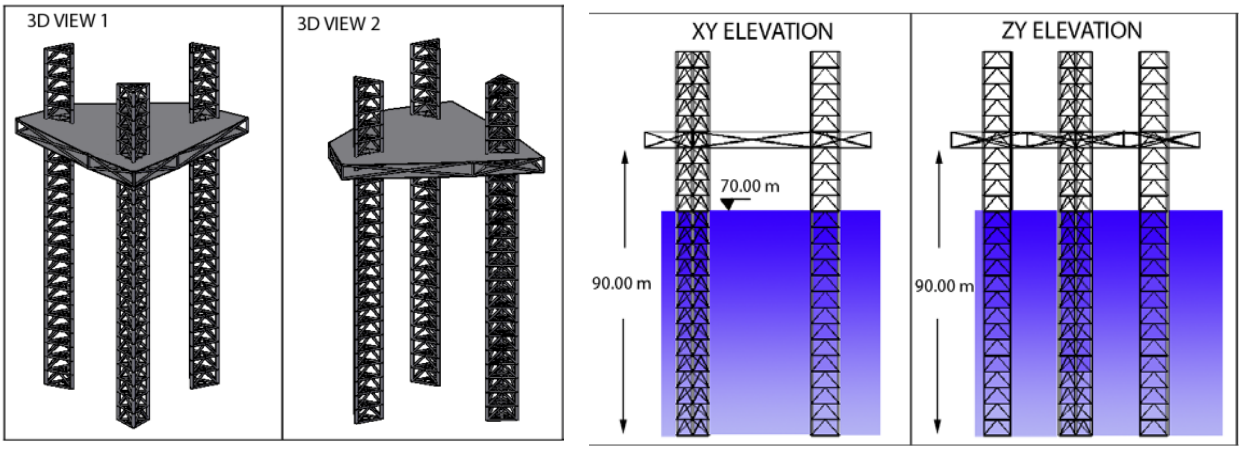

Figure 5. Various view angles of the 3D Jack-Up FE model considered

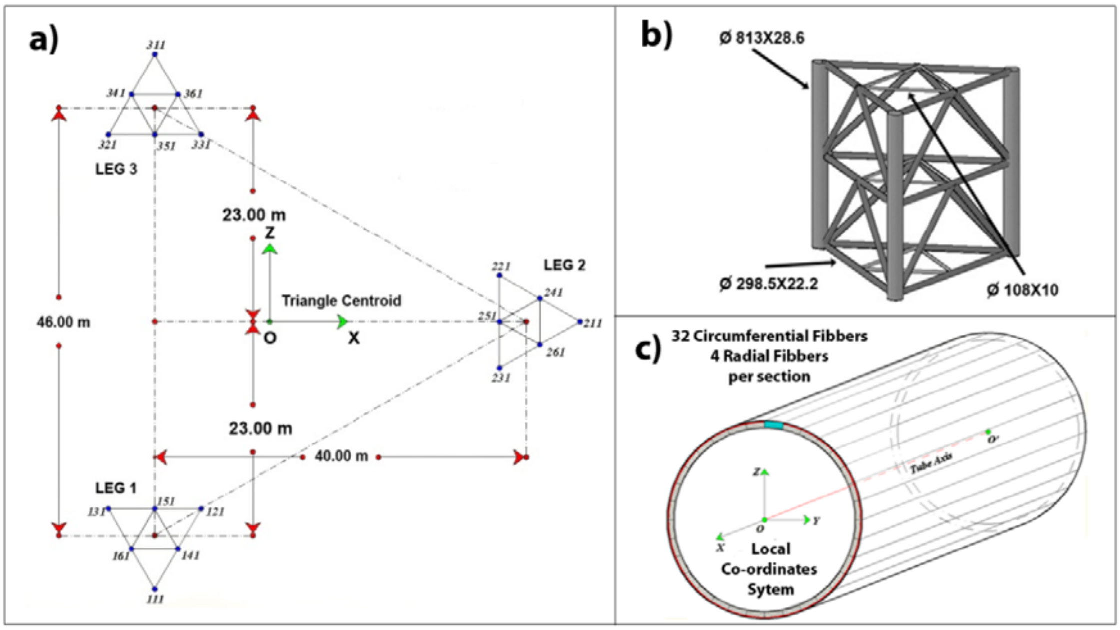

Figure 6. (a) Plan view geometry, (b) members of truss legs, (c) FE modelling of members.

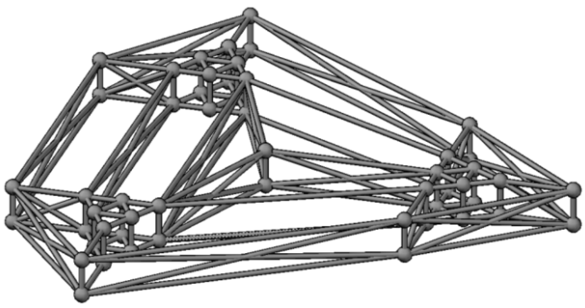

Figure 7. Three dimensional rendered view of the adopted deck model. Mass is lumped at nodes (spheres)

A dual-layer deck modelling approach consistent with the typical jack-up design has been followed to capture the expected linear rigid diaphragmatic action of the deck to lateral seismic (inertial) dynamic loads. In particular, each "deck layer" is interconnected with rigid beam elements in flexure and in axial action as shown in Figure6. Further, rigid connectivity between leg and deck nodes is assumed and the deck mass is assumed to be equally distributed and lumped at both layers of the deck nodes (Figure 6).

Lateral load resistance of jack-up units is provided by the three truss legs which, in combination with the deck, form an "inverted pendulum" type structural system. Truss members of the legs are expected to yield for high levels of seismic severity (e.g. [6,24]). In this regard, they need to be modelled to account for local material and geometric nonlinear behaviour in a more accurate manner than the one achieved using a simplistic "stick" model (legs modelled by beam elements), commonly used in linear dynamic analysis applications (e.g. [5,33]). The steel truss legs of the herein considered jack-up structure comprise tubular steel members (Figure 5(b)) whose cross-section dimensions, mechanical, and material properties are given in Table 1. These tubular members are modelled using the fibber element of OpenSEES [32] to achieve distributed plasticity FE modelling (see also [22]). Specifically, 32 circumferential (Figure 5(c)) and 4 radial fiber divisions are considered with 5 integration points. The assumed material law follows a bilinear perfectly elasto-plastic stress-strain relationship. Further, "P-δ" non-linear geometrical effects are also taken into account.

Table 1. Mechanical and material properties of jack-up leg structural members of Figure 5(b)

Member cross-section	LCHS: 813x28.6 (mm)	CHS: 98.5x22.2 (mm)	CHS: 108x10 (mm)
Young's modulus (GPa)	210	210	210
Yield stress (MPa)	686	353	353
Shear modulus (GPa)	80.77	80.77	80.77
Axial rigidity (MN)	3.0125×10^4	8.414×10^3	1.359×10^3
Torsional rigidity (MNm^2)	7.394×10^4	2.677×10^2	5.555
Flexural rigidity (MNm^2)	113.98×10^4	217.62×10^2	51.23

3.2 Soil-Structure Interaction

The fixity conditions of the spudcan to the seabed constitute a major concern in jack-up platforms assessment and design. Various models accounting for the dynamic nonlinear behaviour of compliant soils and the associated energy dissipation due to hysteretic behaviour and radiation damping have been proposed in the related literature (e.g. [3,6,34]). Such models can be readily incorporated in the OpenSEES platform to perform IDA but their calibration requires site-specific geotechnical investigations. Since the herein study does not relate to any particular site, a relatively simple dynamic soil-structure interaction model is adopted considering translational springs along three orthogonal degrees of freedom connected to all nodes at the foundation level (Figures 8(a) and (b)). The latter is assumed to coincide with the sea bed level and, thus, spudcans are not explicitly modelled. The considered springs follow a bilinear softening hysteretic force displacement relationship to account for energy dissipation during dynamic cycles of loading-unloading-reloading as shown in Figure 8.

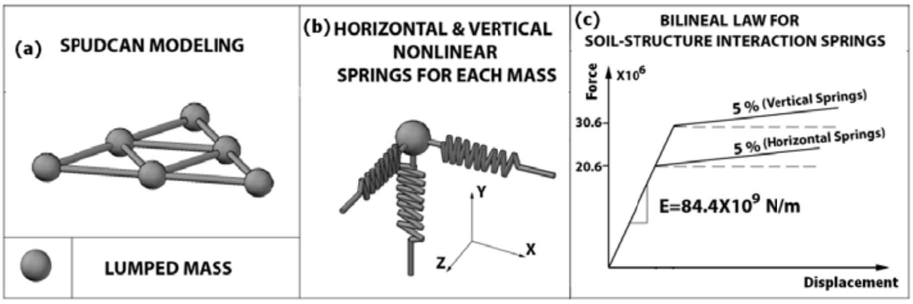

Figure 8. (a) Foundation modelling (last horizontal "layer" of each leg at sea bed level), (b) Springs modelling soil-structure interaction at each spudcan node, (c) Assumed spring force-deformation relationship at foundation.

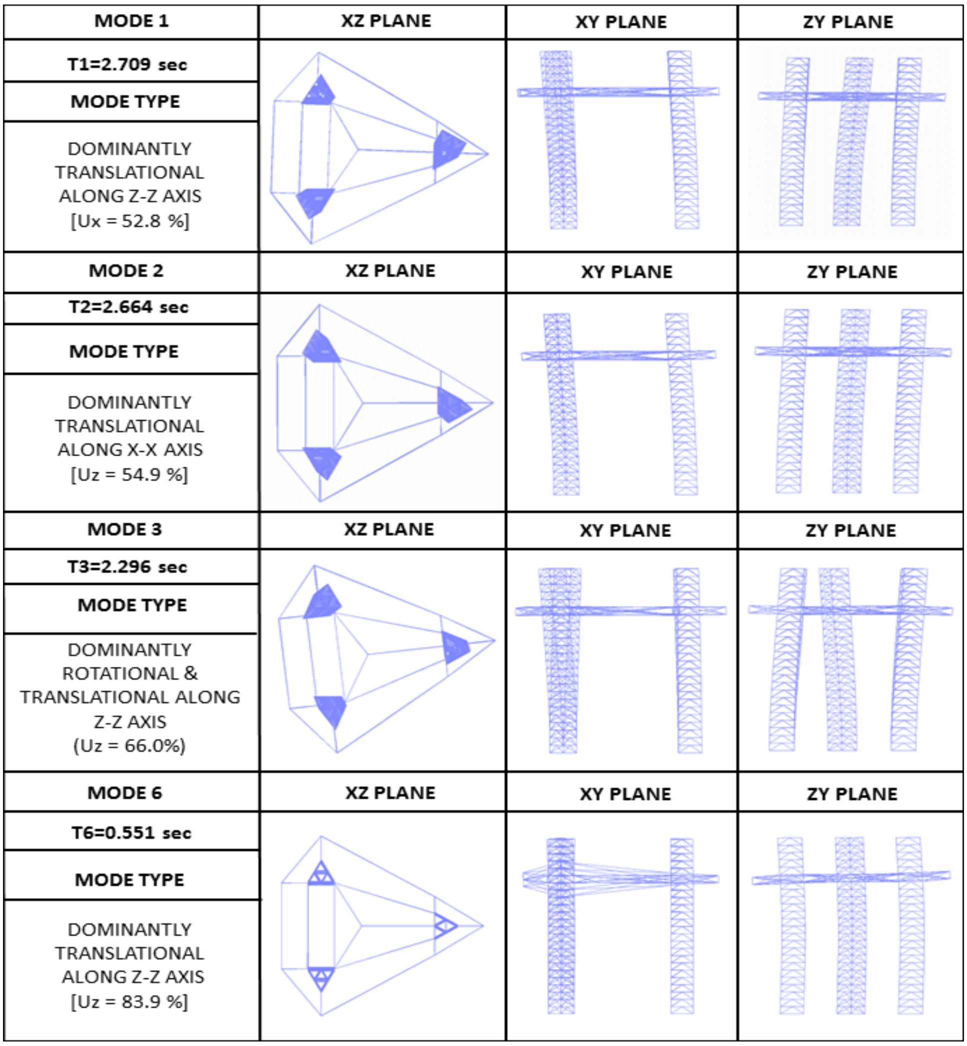

Figure 9. Selective modal analysis results.

3.3 Modal Properties

The considered jack-up platform is assumed to operate at 70m sea depth while the deck is elevated at 90m leaving an air-gap of 20m (Figure 4). The deck mass is taken equal to 11.400 tn and the mass per truss leg is 606 tn. The deck and leg mass is lumped at the FE model nodes. To account for the added hydrodynamic mass as a consequence of oscillatory motion of the submerged nodes within the body of water, the corresponding nodal masses where increased by 80% (see e.g. [3]).

Furthermore, 5% mass and stiffness proportional (Rayleigh) viscous damping is considered in all ensuing analyses. This accounts for 3% contribution of structural damping typically assumed for bare steel structures plus 2% hydrodynamic damping associated with the drag forces acting to the submerged part of the truss legs during their vibratory motion (see e.g. [3]).

Certain results derived from standard modal analysis of the considered 3D FE model are included in Figure10. The first two modes of the vibration are dominantly translational along the orthogonal axes Z-Z and X-X of the adopted global co-ordinate system (Figure 5(a)), having relatively closely-spaced natural periods. The third mode is dominantly rotational about the Y-Y axis. Most of the higher modes of vibration are associated with local deck or leg dynamic degrees of freedom of limited usefulness in characterizing the global dynamic behaviour of the considered structure to inertial horizontal forces.

4. Response of jack-up platform to a near-fault ground motion for various incident angles

In this section the influence of near-fault forward directivity effects to peak inelastic seismic demands of the typical jack-up structure presented in the previous section is assessed. This is accomplished by undertaking IDA for a horizontal orthogonal pair of ground motion (GM) components recorded at Petrolia station during the Cape Mendocino (1992) earthquake event of magnitude M_w= 7 with an epicentre close to the coast of Northern California. Certain properties of the considered GMs are reported in Table 2. Further, Figure 11 plots the time-history traces and linear 5% damped response spectra of the two GM components. The faul-normal component bears a “forward-directivity signature” as classified by Baker [12] using the wavelet transform in conjunction with the fourth-order Daubechies wavelet family. This component has been selected to be used for the purposes of this study as its identified velocity pulse period T_p= 3s is close to and slightly longer than the fundamental natural period of the adopted jack-up FE model equal to 2.71s (Figure 10). Thus, resonance considerations and the fact that the expected material yielding and/or soil-structure interaction effects will cause a transient period elongation of the structure (see e.g. [35]) suggest that the considered component can be viewed as a “worst-case scenario earthquake” for the particular structure. The dominant velocity pulse is highlighted in red on the velocity time-history of the pulse-like component in Figure 11. In alignment with seismological considerations (e.g. [15]), the fault parallel component does not contain such a dominant low frequency pulse at the onset of the ground motion. Further, the response spectra of Figure 11 confirm that the fault-normal component poses higher demands to flexible linear oscillators of longer than 1s natural period.

Following the usual steps in undertaking IDA, the two GMs are applied separately to perform dynamic response history analysis (RHA). Since a 3D FE model is considered and given that some freedom exists in choosing the in-plan orientation of a deployed jack-up platform compared to a local (known) seismic fault, it is deemed important to study the influence of the seismic waves incident angle. In this regard, it is noted that Athanatopoulou [36] has derived analytical formulae to determine the critical angle of incidence (the one that maximizes the peak value of structural response quantities) for linear RHA. However, for the case of nonlinear RHA recent studies, such as those by Rigato and Medina [37] and Goda [38], suggest that the angle of incidence influence significantly structural seismic response, while there can be no closed-form relationships for the critical angle of incidence which is structure- and GM-dependent. To this end, the considered GMs are applied along five different incident angles at 0°, 45°, 90°, 135°, and 180° (the latter results in "sign reversal" compared to 0°) with regards to the adopted global coordinate system as shown in Figure 11. In this manner, an equally-spaced discretization of the incident angle domain is established (see also [26]), which is restricted within the range 0° to 180°. This range is dictated by the fact that the considered jack-up FE model possesses an axis of symmetry (global X-X axis) under the assumption of uniformly distributed deck mass.

Table 2. Properties of the considered strong ground motion components recorded at Petrolia station during the Cape Mendocino (1992) seismic event (retrieved from http://peer.berkeley.edu/peer_ground_motion_database)

Epicentral distance (km)	Distance from fault (km)	Component	PGA (g)	PGV (cm/s)	T_p(s)
4.5	8.2	Fault parallel	0.59	48.02	0.95$
		Fault normal	0.66	89.54	3.0*

$As reported in the PEER database; *As reported in Baker [12]

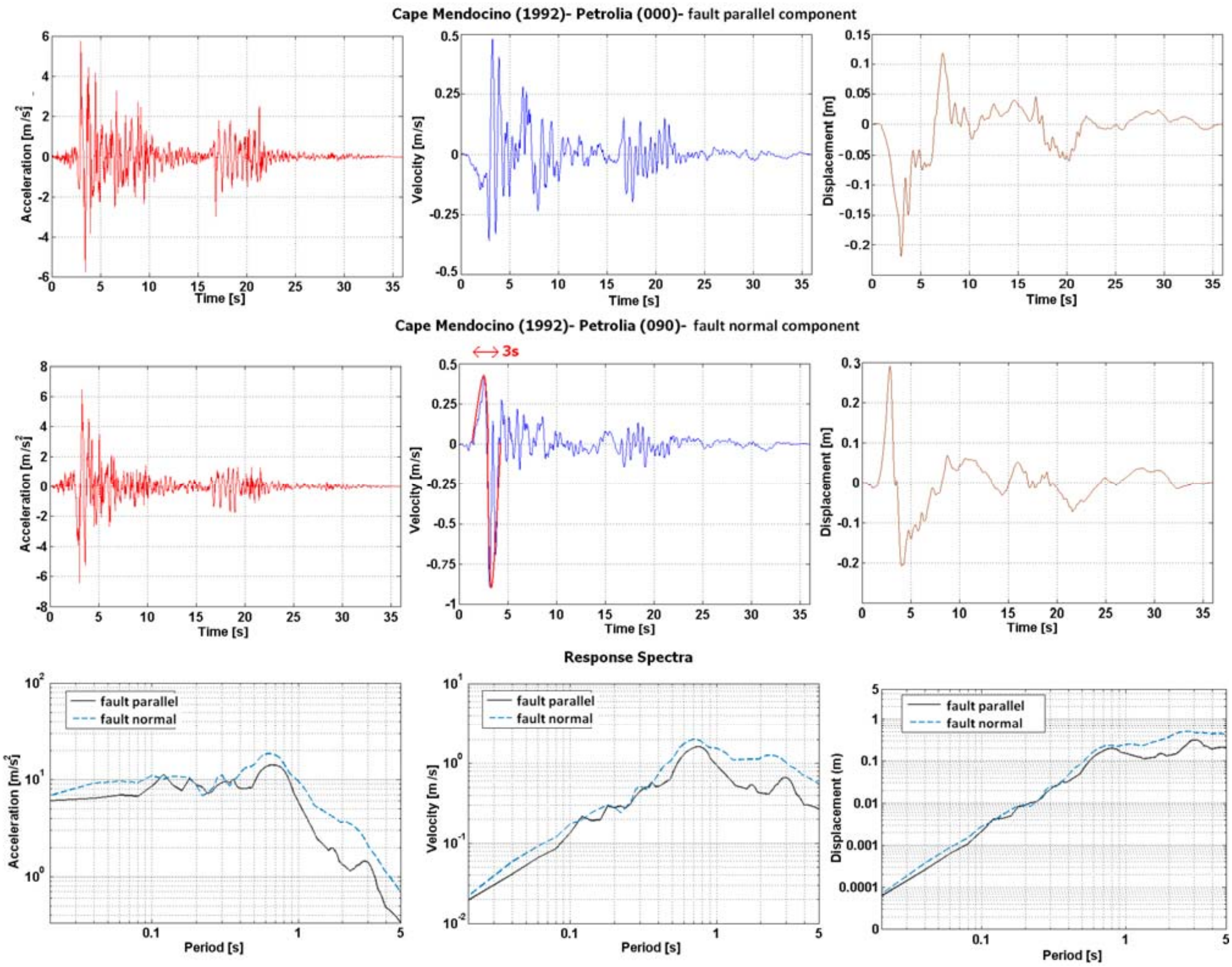

Figure 10. Time history traces of acceleration, velocity, and displacement, and response spectra in terms of pseudo-acceleration, pseudo-velocity, and relative displacement for the ground motions of Table 2.

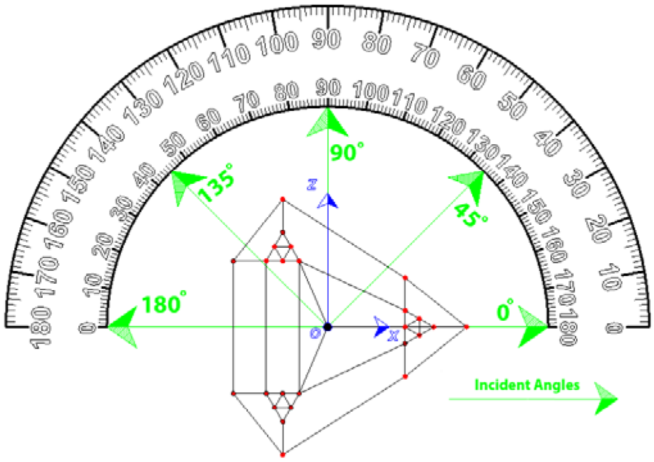

Figure 11. Considered incident angles of the seismic input action.

4.1 IDA curves for fault normal and fault parallel components

The derived IDA curves for the GMs of Table 2 along all 5 incident angles of Figure 11 are plotted in Figure 12. The peak ground velocity (PGV) has been adopted as the IM and the "inter-storey" drift defined in Figure 4 as the EDP for reasons detailed in section 2. The acceleration traces of the GMs are scaled in the time domain to span the full range of elastic response behaviour to global collapse in terms of the PGV. The "collapse points" are highlighted by shadowed areas around the last determined IDA curve point (marked with an "x"). The derived curves exhibit all three response phases (linear-inelastic-collapse) commonly observed in IDA, though these phases are easier identified for the fault parallel GM. In fact, the shape of the IDA curves for the fault parallel GM resembles the one observed in the case of far-field accelerograms [24].

In the herein derived IDA curves, the linear phase corresponds to a small fraction of the entire response range indicative of the high ductility capacity (ability to deform inelastically without pre-mature global instability) of the truss type steel legs. For the considered GMs, elastic behaviour is exhibited for PGV<10 (cm/s). By increasing the PGV beyond 10cm/s, several hardening (slope of IDA curve greater than the one corresponding to the linear phase) and softening (slope of IDA curve smaller than the one corresponding to the linear phase) phases on the IDA curves are observed related with the load path redistribution and/or structural yielding occur. These changes in the stiffness can be readily captured by the considered 3D FE model. Evidently the pulse-like (fault normal) accelerogram induce far greater inelastic deformations for the same PGV compared to the fault parallel component. It further "drives" the FE model to collapse "faster" than the fault parallel component as it is indicated by the proximity of the corresponding IDA curve to the EDP (horizontal) axis. This can be readily justified by considering the spectral ordinates of the two orthogonal components about the fundamental natural period of the structure (2.7s) shown in Figure 10: the fault normal component attains significantly higher spectral values for both stiffer and more flexible systems compared to the oscillator with natural period 2.7s.

4.2 Impact of the seismic incident angle

Table 3 collects the collapse capacity values in terms of the chosen EDP (maximum of the peak values of "inter-storey drifts" attained across the height of all legs along the direction of the seismic action) for all seismic wave incident angles considered. Taking as the reference value of the EDP the one attained at 0° (see also Figure 11), a significantly higher spread in the collapse capacity values is observed for the fault-normal component vis-à-vis the fault-parallel component which suggests that the angle of incidence of seismic waves influences considerably the inelastic seismic demands of jack-up platforms subject to pulse-like ground motions. Noticeably, at 45° incident angle, an increase of 63% in the collapse capacity compared to the 0° is observed for the fault-normal component. However, for the same angle of incidence a decrease in the collapse capacity of 15.3% is noted for the fault-parallel component. The latter observation suggests that the influence of the incident angle to the collapse capacity is GM-dependent.

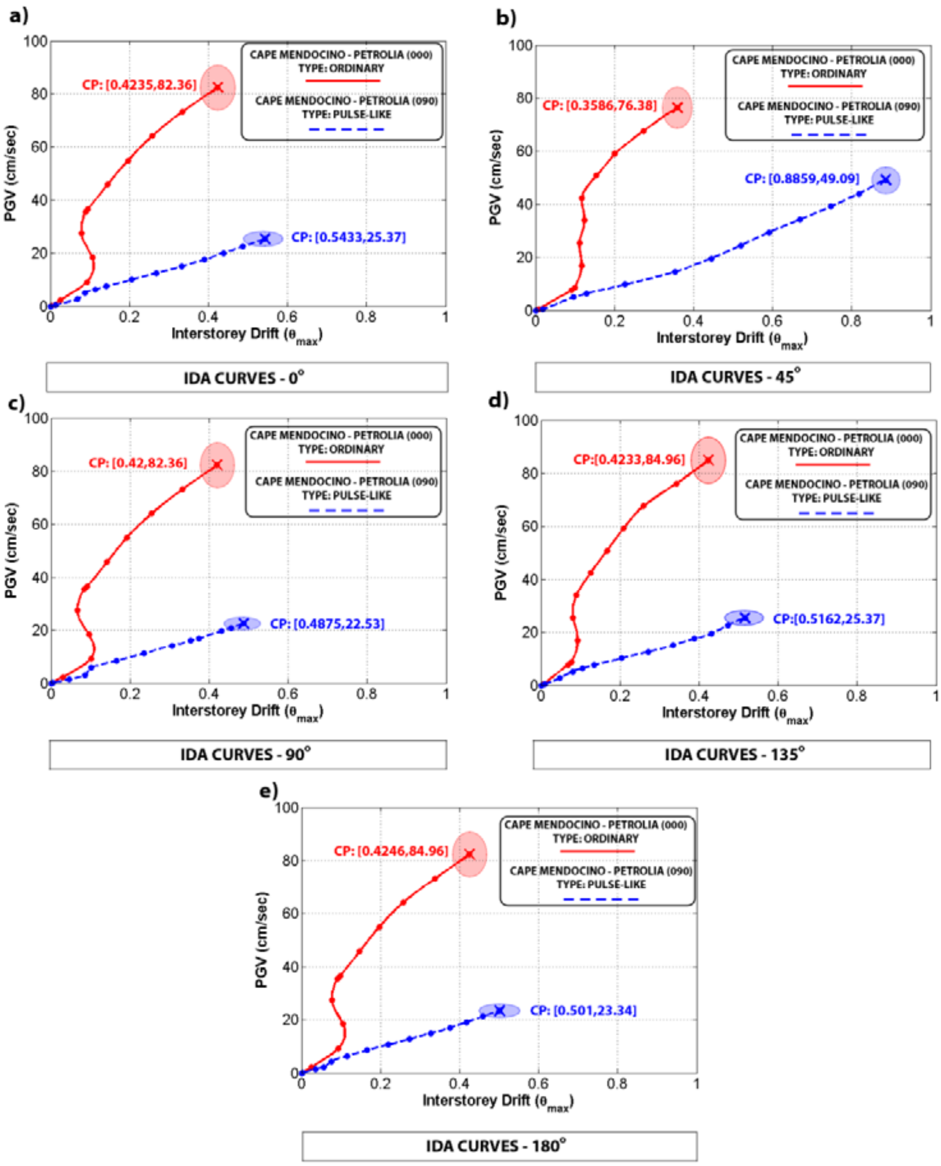

Figure 12. IDA curves for various incident angles.

Moreover, the influence of the incident angle of seismic waves across all response phases (linear-inelastic-collapse) can be graphically visualized by contour and 3D plots of surfaces generated using (linear) interpolation between IDA curve points. Such plots are shown in Figure 13. It is readily observed that the EDP "distribution" along different IM levels and incident angles for the fault-parallel accelerogram is much more even than for the fault-normal component. Further, the enhanced performance of the structure for the fault-normal component acting along a 45° incident angle is readily seen across all IM levels.

Table 3. Comparison of the collapse structural demand for various incident angles.

CAPE MENDOCINNO - PETROLIA (000) – Fault parallel component					
Angle (degrees)	0 (reference)	45	90	135	180 (sign reversal)
Collapse EDP (Θ_{max})	0.4235	0.3586	0.42	0.4233	0.4246
Difference from reference	-	- 0.0649	- 0.0035	- 0.0002	+ 0.0011
Percentage of difference	-	**- 15.32%**	- 0.83%	- 0.05%	+ 0.26%
CAPE MENDOCINNO - PETROLIA (090) –Fault normal component					
Angle (degrees)	0 (reference)	45	90	135	180 (sign reversal)
Collapse EDP (Θ_{max})	0.5433	0.8859	0.4875	0.5162	0.501
Difference from reference	-	+ 0.3426	- 0.0558	- 0.0271	- 0.0423
Percentage of difference	-	**+ 63.06%**	- 10.27%	- 4.99%	- 7.79%

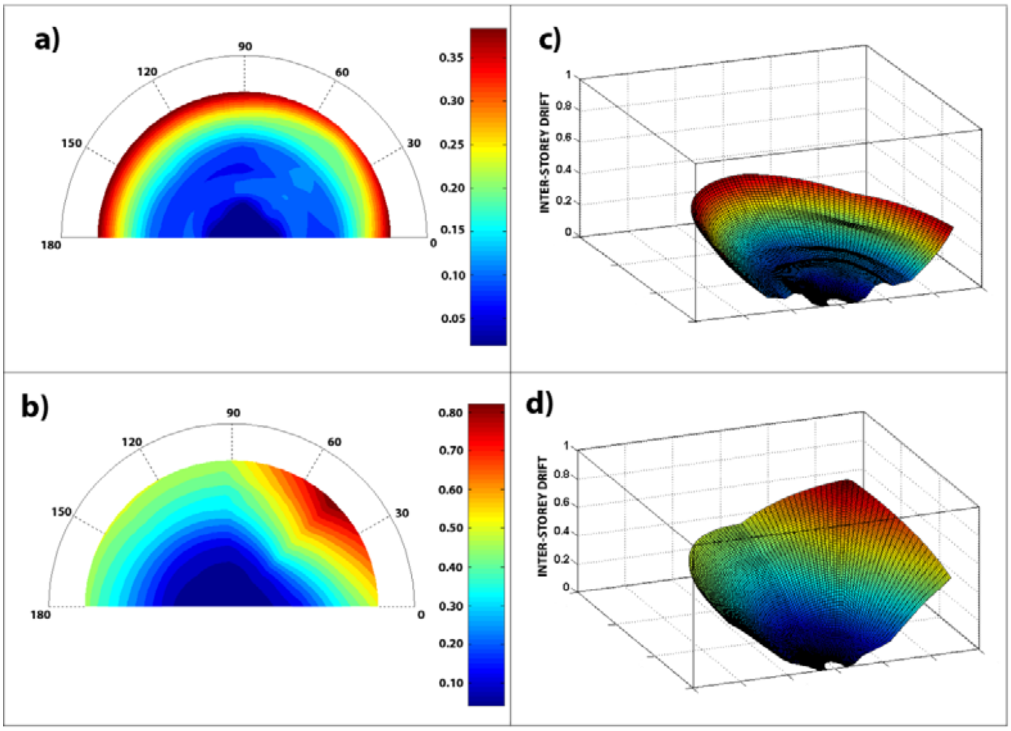

Figure 13. IM-incident angle- EDP plots. a) Contour EDP plot for the fault-parallel component b) Contour EDP plot for the fault-normal component c) EDP response surface for the fault-parallel component d EDP response surface for the fault-normal component.

5. Concluding remarks

The influence of near-fault effects and of the incident angle of earthquake waves to the seismic response of a typical jack-up offshore platform has been investigated by means of incremental dynamic analysis (IDA) involving a three dimensional (3D) distributed plasticity finite element model. Two horizontal orthogonal strong ground motion components of a judicially chosen near-fault seismic record have been considered to represent the input seismic action along five different incident angles. The fault-normal component exhibits a prominent forward-directivity velocity pulse (pulse-like) whose period lies close to the fundamental natural period of the considered structure following a "worst case scenario" approach, while the fault-parallel component does not include such a pulse. Results in the form of IDA curves (IM: peak ground velocity versus EDP: peak inter-storey drift along the direction of the seismic action) were derived for both earthquake components and for all considered incident angles. These curves were further collectively considered to derive 3D surfaces of EDP-IM-incident angle using linear interpolation to facilitate the interpretation of numerical data. The thus derived data demonstrate that the "pulse-like" fault normal component poses much higher seismic demands to the "prototype" jack-up structure considered compared to the fault parallel component. Further, significant variations in the collapse resistance values have been observed when considering the same components acting along different incident angles, especially for the "critical" fault normal component. In this regard, from the limited amount of numerical results herein considered, it can be concluded that the combined influence of near-fault forward directivity effects and the incident angle of seismic waves to the seismic hazard posed to jack-up structures is considerable and needs to be appropriately taken into account when operating close to active seismic faults. Further on-going research by the authors is channelled towards collecting a statistically significant collection of numerical data accounting for the inherent uncertainty of the near-fault seismic input action and treating important structural parameters affecting the dynamic properties of jack-up structures as random variables within a Monte Carlo context. In this manner, recommendations on the orientation of deployment of jack-up structures with respect to neighbouring active seismic faults that minimize the seismic risk as a function of various dynamic structural properties can be drawn.

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