Ocean Engineering 85 (2014) 1–11

Contents lists available at ScienceDirect

Ocean Engineering

journal homepage: www.elsevier.com/locate/oceaneng

Sensitivity analysis of pile-founded fixed steel jacket platforms subjected to seismic loads

Mohamed Nour El-Din [a], Jinkoo Kim [b,*]

[a] Samsung Engineering, Co., Ltd., Republic of Korea
[b] Department of Civil and Architectural Engineering, Sungkyunkwan University, Suwon, Republic of Korea

ARTICLE INFO

Article history:
Received 12 January 2013
Accepted 5 April 2014

Keywords:
Seismic performance
Fixed jacket platforms
Offshore structures
Sensitivity analysis
Tornado diagram
First-order second moment

ABSTRACT

The sensitivity of the seismic response parameters to the uncertain modeling variables of pile-founded fixed steel jacket platforms are investigated using the Tornado diagram and the first-order second-moment techniques. The effects of both aleatory and epistemic uncertainty on seismic response parameters have been investigated for an existing offshore platform. The sources of uncertainty considered in the present study are categorized into three different categories: the uncertainties associated with the soil–pile modeling parameters in clay soil, the platform jacket structure modeling parameters, and the uncertainties related to ground motion excitations. It has been found that the variability in parameters such as yield strength or pile bearing capacity has little effect on the seismic response parameters considered, whereas the global structural response is highly affected by the ground motion uncertainty. Also, some uncertainty in soil–pile property such as soil–pile friction capacity has a significant impact on the response parameters and should be carefully modeled. Based on the results, it is highlighted that which uncertain parameters should be considered carefully and which can be assumed with reasonable engineering judgment during the early structural design stage of fixed steel jacket platforms.

1. Introduction

Performance-based earthquake engineering (PBEE) requires accurate estimation of the structural seismic demands. One of the factors that decrease this accuracy is the uncertainties in seismic responses caused by uncertainties associated with the input parameters. Seismic responses of offshore platforms are affected by various uncertain input parameters. Through sensitivity analysis based on reliable data the expected ranges of structural responses can be identified.

Sources of uncertainty affecting structural performance are often characterized as either aleatoric or epistemic in nature. Aleatoric uncertainty stems from the unpredictable nature of events, whereas epistemic uncertainty is due to incomplete data, ignorance, or modeling assumptions (Padgett and DesRoches, 2007). In general structures, sources of uncertainty include those which affect both the structural capacity and demand including the seismic forces, material properties, and geometry. In fixed type offshore platforms another important source of uncertainty is the soil–pile properties. The level of nominal capacity required for a system will be increased with the higher uncertainty in either seismic demand or capacity. Reducing the number of uncertain variables leads to decreasing the required level of the nominal capacity of the structure under investigation and hence reducing the cost.

Sensitivity of the seismic demand or estimated fragility to varying parameters in a range of structural systems has been assessed in various studies. Kwon and Elnashai (2006) studied the effects of ground motion input and material variability on the vulnerability curves of a three-story RC structure using nine sets of ground motions. Wang and Foliente (2006) found that uncertainties due to ground motion and structural modeling are the major sources for increase in estimated structural demand for Seismic responses and reliability of a L-shaped wood frame building. Song and Ellingwood (1999) studied four welded special moment-resisting frames of different sizes and configurations that suffered connection damage during the earthquake and evaluated the seismic performance using both deterministic and stochastic approaches. Kim et al. (2011) studied the sensitivity of design parameters of steel buildings subjected to progressive collapse. Nielson and DesRoches (2006) performed a seismic evaluation of a typical configuration for a multi-span simply supported steel girder bridge for an approximate hazard level of 2% in 50 years. Padgett and DesRoches (2007) studied the sensitivity of a multi-span simply supported steel girder bridge. Jalayer et al. (2010) characterize the uncertainties in material properties and in construction details and propagate them to estimate the structural

* Corresponding author. Tel.: +82 31 290 7563; fax: +82 290 7570.
E-mail address: jkim12@skku.edu (J. Kim).

http://dx.doi.org/10.1016/j.oceaneng.2014.04.008
0029-8018/

performance conditional on code-based seismic demand and capacity definitions. Rota et al. (2010) proposed a new analytical approach based on nonlinear stochastic analyses of building prototypes for the derivation of fragility curves for masonry buildings. Fragiadakis and Vamvatsikos (2010) introduced approximate methods based on the static pushover to estimate the seismic performance uncertainty of structures having non-deterministic modeling parameter. Celarec et al. (2012) investigated the sensitivity of seismic response parameters to the uncertain modeling variables of four infilled RC frames using pushover analysis. Dolsek (2012) proposed a simplified method for seismic risk assessment of buildings with consideration of aleatory and epistemic uncertainty. The method involves a nonlinear static analysis of a set of structural models, which is defined by utilizing Latin hypercube sampling, and non-linear dynamic analyses of equivalent single degree-of-freedom models. Recently Celarec and Dolsek (2013) used simplified procedures for the estimation of seismic response parameters by considering the epistemic uncertainties for an older reinforced concrete frame, and for two contemporary reinforced concrete structures. The simplifications in the procedure are associated with a simplified nonlinear method and models for the assessment of the seismic performance of the structure, whereas the effects of the epistemic uncertainty are treated by using the first-order-second-moment (FOSM) method and the Latin Hypercube Sampling (LHS) technique.

Pile-founded offshore platforms are now being installed in seismically active and environmentally sensitive regions (Yasseri and Ossei, 2004). Failure of pile-founded offshore structure may affect not only the oil and gas production activity or the safety and serviceability of the platform but also it may have worse environmental impact. However, few studies have considered the impact of uncertainty inherent to offshore structures, which have the common complexity of geometric uncertainties found in common building structures in addition to the complexity of parameters uncertainties inherent in soil–pile structure interaction. Overall structural response and capacity of pile-founded offshore platform greatly depends on the member behavior in the nonlinear range of deformation and the non-linear interaction of the foundation with the soil. In order to identify the impact on seismic response of offshore platform, sensitivity analysis is required to investigate the contribution of those uncertain input parameters including those from soil–pile interaction on the platform overall seismic performance.

This study presents a seismic sensitivity analysis of a fixed type steel offshore platform. It addresses the important uncertain modeling parameters that may contribute significantly to the overall performance uncertainty of an offshore platform designed according to the provisions of the API, American Petroleum Institute Recommended Practice for Planning (2000). After that, a simple deterministic sensitivity methodology has been used to investigate the effect of each uncertain input parameter on some engineering demand parameters (EDP) such as the maximum top displacement (MTD) and the maximum inter story drift ratio (MIDR) of the jacket structure.

2. Sensitivity analysis methods applied

In the present study, two different methods have been adopted in the sensitivity analysis of the offshore platform structure under investigation using nonlinear dynamic analysis. These methods are based on the probability theories which are the Tornado Diagram Analysis (TDA) and the First-Order Second Moment (FOSM) methods. In TDA, the upper and lower bounds of a random variable are selected and the corresponding structural responses are obtained. The difference between such structural responses, referred to as swing, is considered as a measure of sensitivity. This method has been applied in the seismic sensitivity analysis of structures in many previous studies, (e.g., Porter et al., 2002; Barbato et al., 2010, and Kim et al., 2011). In the FOSM method, the mean and the standard deviation of input parameters are predetermined and those of the structural response are obtained through simple computation. Ibarra (2003) evaluated the collapse capacity uncertainty of frame structures under seismic excitation using FOSM principles verified through the Monte Carlo simulation method. Lee and Mosalam (2005) have also used FOSM to determine the response uncertainty of a reinforced-concrete (RC) shear wall structure to several modeling parameters. Haselton (2006) has studied the effects of modeling uncertainties on the collapse capacity of reinforced concrete frames designed for a high seismic region in California using the FOSM reliability approach. Also, Baker and Cornell (2003, 2008) have used the FOSM method in combination with numerical integration for the propagation of uncertainties in probabilistic seismic loss estimation.

3. Uncertain variables considered in the analysis

The sources of uncertainty considered in the present study consist of three different categories. The first is the uncertainties associated with soil–pile modeling parameters including axial pile–soil friction, the pile end bearing, the effect of time since the pile was driven, and the cyclic nature of loading during the pile driving. The second source of uncertainty is related to the platform jacket structure modeling parameters including structural mass, damping ratio, elements yield strength (F_y), Young's modulus (E), and the force–deformation relationship of element plastic hinges. The variation of plastic hinge property is obtained by scaling every force and deformation value on the force–deformation relationship by multiplying a single, random variable. The third is associated with the seismic excitation including ground motion intensity and ground motion profile. The variation of ground motion profile is considered by performing a set of structural analysis using a scaled ground motion profile and sorting the set of ground motion profile with respect to the magnitude of EDP values.

Based on the ISO Code 19902 (ISO, 2003) for Fixed Steel Offshore Structures, a reliability analysis has been carried out for pile axial capacity to assess the effect of different environmental load factors on foundation reliability. Statistical modelings for pile friction and end-bearing capacities have been assessed based on large scale tests (ISO, 2003). The axial capacity of a piled foundation in clay soils depends on the shaft friction, the end bearing, the set-up or effect of time since the pile is driven or last disturbed, and the cyclic nature of the loading. The capacity prediction equation assumed in the mentioned reliability analysis study for piles in clay soil under compression is as follows (ISO, 2003):

$$Q_d = (Q_f X_{friction} X_{delay} + Q_p X_{bearing}) X_{cyclic} \quad (1)$$

where Q_d is the pile ultimate bearing capacity, Q_f is the pile skin friction resistance; Q_p is the pile total end bearing; and $X_{friction}$, X_{delay}, $X_{bearing}$, and X_{cyclic} are the random variables of the shaft friction, the set-up or effect of time since the pile is driven or last disturbed, the end bearing, and the cyclic nature of the loading, respectively. The statistical properties of random variables of clay soil are listed in Table 1.

The statistical properties of structural modeling parameters are listed in Table 2. All variables are assumed to be uncorrelated. The uncertainty in dead loads including buoyancy in typical offshore platforms arises from factors such as rolling tolerances, fabrication aids, paint and fire protection, approximation in weight take-off, marine growth, etc. Based on the ISO (2003) the uncertainty in dead load can be modeled by a normal distribution with a mean bias of 1.0 and standard deviation of 0.06. Damping from water-interaction effects and foundation and structure related energy dissipation may be reasonably assumed to be in the range of 2–5%

Table 1
Statistical properties of random variables associated with normally consolidated clay soil.

Variables	Distribution	Mean bias	Standard deviation	Source of data
Friction ($X_{friction}$)	Lognormal	0.73	0.19	Smith et al. (1998)
Bearing ($X_{bearing}$)	Lognormal	0.91	0.43	Smith et al. (1998)
Delay(X_{delay})	Lognormal	1.00	0.07	ISO (2003)
Cyclic(X_{cyclic})	Lognormal	0.86	0.02	ISO (2003)

Table 2
Statistical properties of structural modeling parameters.

Variable	Distribution	Mean bias	Standard deviation	Source of data
Dead load	Normal	1.0	0.06	ISO (2003)
Yield stress	Lognormal	1.1	0.05	ISO (2003)
Young's modulus	Normal	1.0	0.05	ISO (2003)
Damping ratio	Lognormal	1.0	0.02	Kim et al. (2011)
Plastic hinge property	Normal	1.0	0.20	Ellingwood et al. (1980)

of the critical (Anagnostopoulos, 1983). API, American Petroleum Institute Recommended Practice for Planning (2000) recommends the same value for analysis of offshore platforms. As the literature for the statistical properties of damping ratio of offshore steel fixed jacket structures is not available, the statistical properties of steel braced frames are used for sensitivity analysis. A lognormal distribution is assumed for damping ratio with a mean value of 5% of critical damping with a coefficient of variance (COV) equal to 0.4 (Kim et al., 2011). Based on the ISO (2003) the distribution of yield stress is modeled with a lognormal distribution with a mean yield stress of 350 N/mm^2 and COV of 5%. The statistical variation of ultimate strength is assumed to have a normal distribution with COV of 0.2 (Ellingwood et al., 1980).

Proper intensity measure is considered a good indicator of EDP response if it is efficient and sufficient to predict the damage (Luco and Cornell, 2001). In the present study, elastic spectral acceleration S_a with 5% of critical damping is selected as a variable parameter to represent the earthquake intensity measure. For seismic analysis the amplitudes of ground acceleration records are generally scaled to the same intensity, but still they are not identical because of the variability in detailed ground motion record profile. Shome and Cornell (1999) have shown that ten to twenty ground motion records are usually enough to provide sufficient accuracy in the estimation of seismic demand for mid-rise buildings. In this study thirty ground motions selected from the PEER ground motions database are used to take the random nature of earthquakes into consideration, such as the effects of the record-to-record (EQ profile) variability and the S_a variability. The ground motion records are generated in such a way that the geometric mean of the response spectra for the records matches the uniform hazard spectrum (UHS) with 2% probability of exceedance in 50 years. This uniform hazard spectrum is used as a representative of the DLE (ductility level earthquake) for the seismic design of the platform. The magnitudes of the selected records range from 7.1 to 7.6 with the closest distances to the normal fault varying from 2.2 to 293 km to obtain ground motions with different characteristics. Table 3 shows the characteristics of the ground motion suite used in the present study, and Fig. 1 shows the response spectra of the generated ground motions and their geometric mean with the target spectrum. The ground motions are assumed to be normally distributed around the

Table 3
Characteristics of the ground motion suit used in the sensitivity analysis.

NGA#[a]	Scale factor	Event	Mag.	R_{jb}[b] (km)	R_{rup}[c] (km)
1785	19.0	Hector Mine	7.13	54.7	54.7
862	7.5	Landers	7.28	54.2	54.2
1153	10.9	Kocaeli – Turkey	7.51	126	127
1800	24.9	Hector Mine	7.13	186.8	186.8
835	7.1	Landers	7.28	135.2	135.2
1163	6.3	Kocaeli – Turkey	7.51	58.3	60
879	3.0	Landers	7.28	2.2	2.2
1604	43.7	Duzce – Turkey	7.14	182.8	183.6
833	6.1	Landers	7.28	144.9	144.9
12	17.2	Kern County	7.36	114.6	117.8
1636	5.4	Manjil – Iran	7.37	50	50
1638	19.6	Manjil – Iran	7.37	174.6	174.6
1805	28.4	Hector Mine	7.13	185	185
853	6.8	Landers	7.28	135.9	135.9
1833	53.6	Hector Mine	7.13	72.9	72.9
1602	1.9	Duzce – Turkey	7.14	12	12
1148	9.4	Kocaeli – Turkey	7.51	10.6	13.5
1799	18.7	Hector Mine	7.13	179.3	179.3
892[d]	11.6	Landers	7.28	163.5	163.5
886	64.3	Landers	7.28	94.5	94.5
1776	16.1	Hector Mine	7.13	56.4	56.4
897	35.4	Landers	7.28	41.4	41.4
1811	11.1	Hector Mine	7.13	91.2	91.2
841	12.4	Landers	7.28	89.7	89.7
889	7.7	Landers	7.28	141.9	141.9
1167	4.8	Kocaeli – Turkey	7.51	145.1	145.1
1634	4.0	Manjil – Iran	7.37	75.6	75.6
861	5.6	Landers	7.28	156	156
1168	28.8	Kocaeli – Turkey	7.51	293.4	293.4
1759	50.7	Hector Mine	7.13	176.6	176.6

[a] Next Generation of Ground-Motion Attenuation Models.
[b] Joyner–Boore distance (km): the horizontal distance to the surface projection of the rupture plane.
[c] Closest distance (km) to the fault rupture plane.
[d] Base record, 50th percentile.

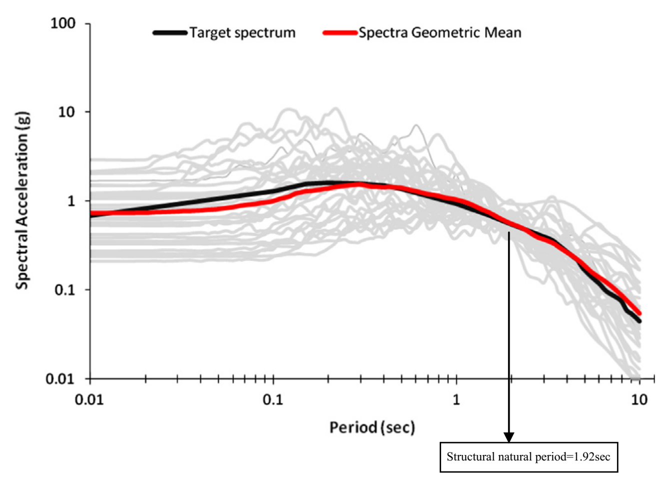

Fig. 1. Response spectra of the generated ground motions along with their geometric mean and the target spectrum (uniform hazard spectrum for 2% probability of exceedance in 50 years of the platform site).

geometric mean spectral acceleration value. The mean spectral acceleration is equal to the spectral acceleration on the target response spectrum of the structure at its natural period. According to the PEER-NGA (2013), the ground motions selected satisfy the magnitude and fault distance while maintaining the relative frequency content of the records. This technique tries to get the target spectrum and the mean spectra close to one another to the possible extent but not to make them identical. Also, the over-estimation in the spectral acceleration in the expected range of the

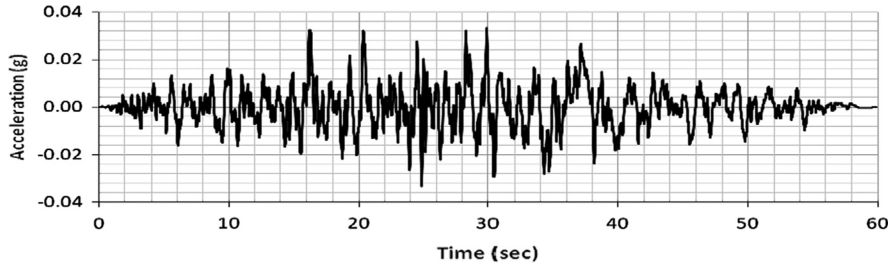

Fig. 2. Time history of the base record (50th percentile) used for S_a variability (NGA# 892, Landers earthquake, unscaled).

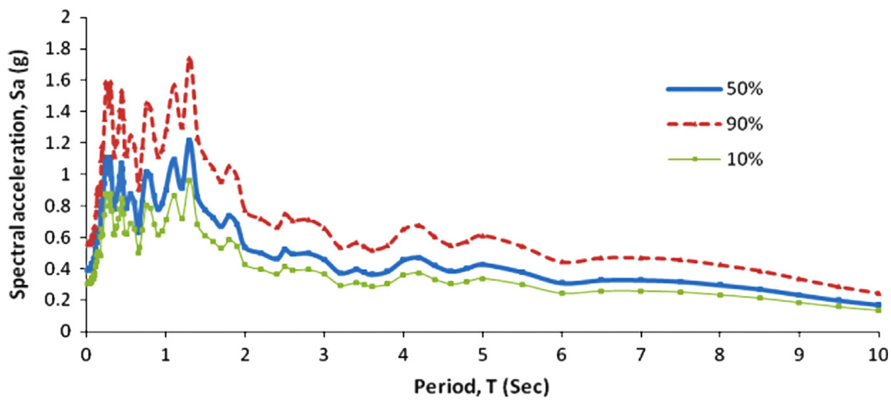

Fig. 3. Response spectra of the earthquakes used to investigate the S_a variability.

elongated natural period, which is in the range of 1.5–2.0 times the structure natural period, is considered relatively small.

The target response spectrum used in the current study is the uniform hazard spectrum (UHS) for 2500 year return period of the platform structure site (PTTEP International, 2010). The record with its response spectrum identical to the target spectrum at the natural period is assumed to be the base record (i.e. 50th percentile). The base record is scaled up by a scale factor I_90, where, I_90 is the S_a at the 90th percentile divided by the S_a at the target. This record is considered as the upper bound or the 90th percentile record of the S_a. The same procedure is followed using the scale factor I_10 for obtaining the lower bound, which is the 10th percentile record. Fig. 2 shows the time history of the base record (50th percentile) which is the Landers earthquake (NGA# 892), and Fig. 3 shows the response spectra of the earthquake used to investigate S_a variability.

Porter et al. (2002) presented two methods for treating with uncertainty in record profiles. The simplest method is chosen in the present study, in which a large number of non-linear dynamic analyses are conducted for the structure, each time using a different ground motion scaled to the intensity of interest. Based on the structural responses, the engineering demand parameters (EDP) of interest are determined and sorted in an ascending or descending order. After that, the lower-bound ground motion record profile would be chosen such that it corresponds to the EDP value that produces the response closest to some predetermined lower fractile such as the 10th percentile. Likewise, the best-estimate and upper-bound record profiles would be those corresponding to the median and the 90th percentile EDP values, respectively. Steps of this method are detailed in Lee and Mosalam (2003).

4. Description of the analysis model structure

The platform model considered in this study comprises of the deck, the jacket and its appurtenances, and the pile foundation. A perspective plot of the platform is shown in Fig. 4, and a frame

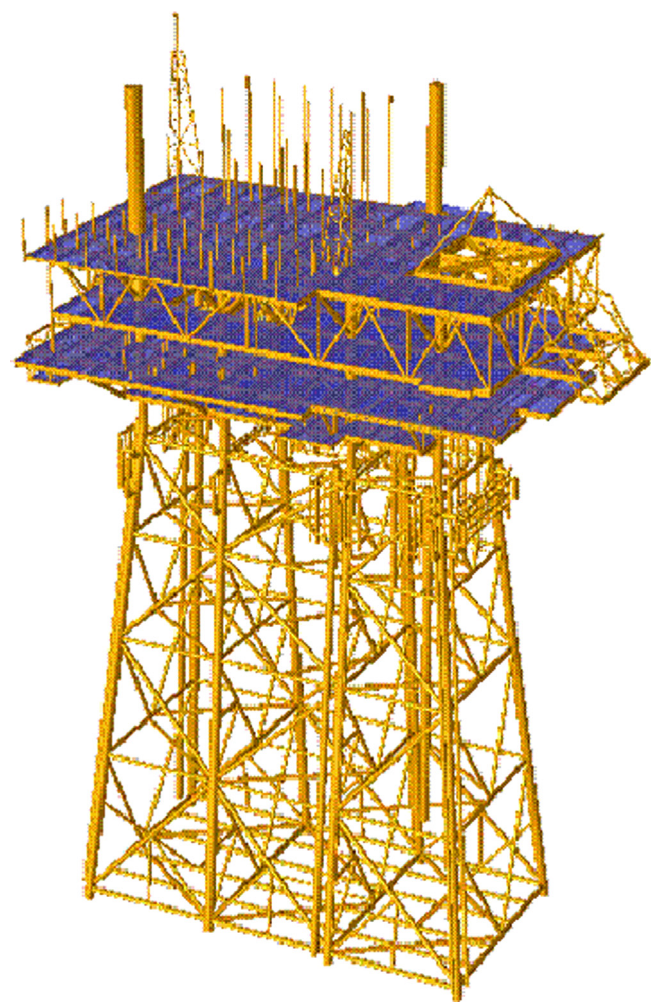

Fig. 4. Perspective plot of the actual platform.

model extracted from the platform structure is shown in Fig. 5. The platform has the topside with four-stories and a four story jacket with total mass of 138,000 t located in the main nodes of the jacket. The appurtenances include the non-structural members such as flooding system, centralizer, pad-eyes, plates and stiffeners, etc. Only the major structural components are included within the analysis model, and the contribution of the conductors to the platforms' stiffness and strength is neglected. The jacket horizontal members are frame elements rigidly connected at the ends. The legs in this platform have the vertical V-shape braces in the short direction. Table 4 shows the dimensions and mass of the model structure, and Table 5 shows the modal periods and characteristics of the first four vibration modes. The mass used in the dynamic analysis consists of the mass of the platform associated with gravity loading defined, the mass of the fluids enclosed in the structure and the appurtenances, and the added mass. The mass of the model frame is applied at each joint, while the mass from the top side structure is applied at the upper two joints of the jacket frame.

The nonlinear dynamic analyses of the model frame structure are carried out using the SAP2000 (2005). A frame element with plastic hinges is chosen from the SAP2000 library to model the nonlinear behavior of platform members. Fig. 6 shows the backbone curve of the inelastic force–displacement (moment–rotation) relationship of a plastic hinge specified in the FEMA (2000). The B-IO range is the first portion after leaving the linear range, and in the C-D range the curve starts to drop abruptly. The modal

Table 4
Characteristics of the platform structure.

Item	Description
Water depth	62.92 m
Jacket height	67.00 m
Jacket plan dimensions	15.2 × 42.7 m
Total numbers of jacket legs	8
Total mass	13,800 t

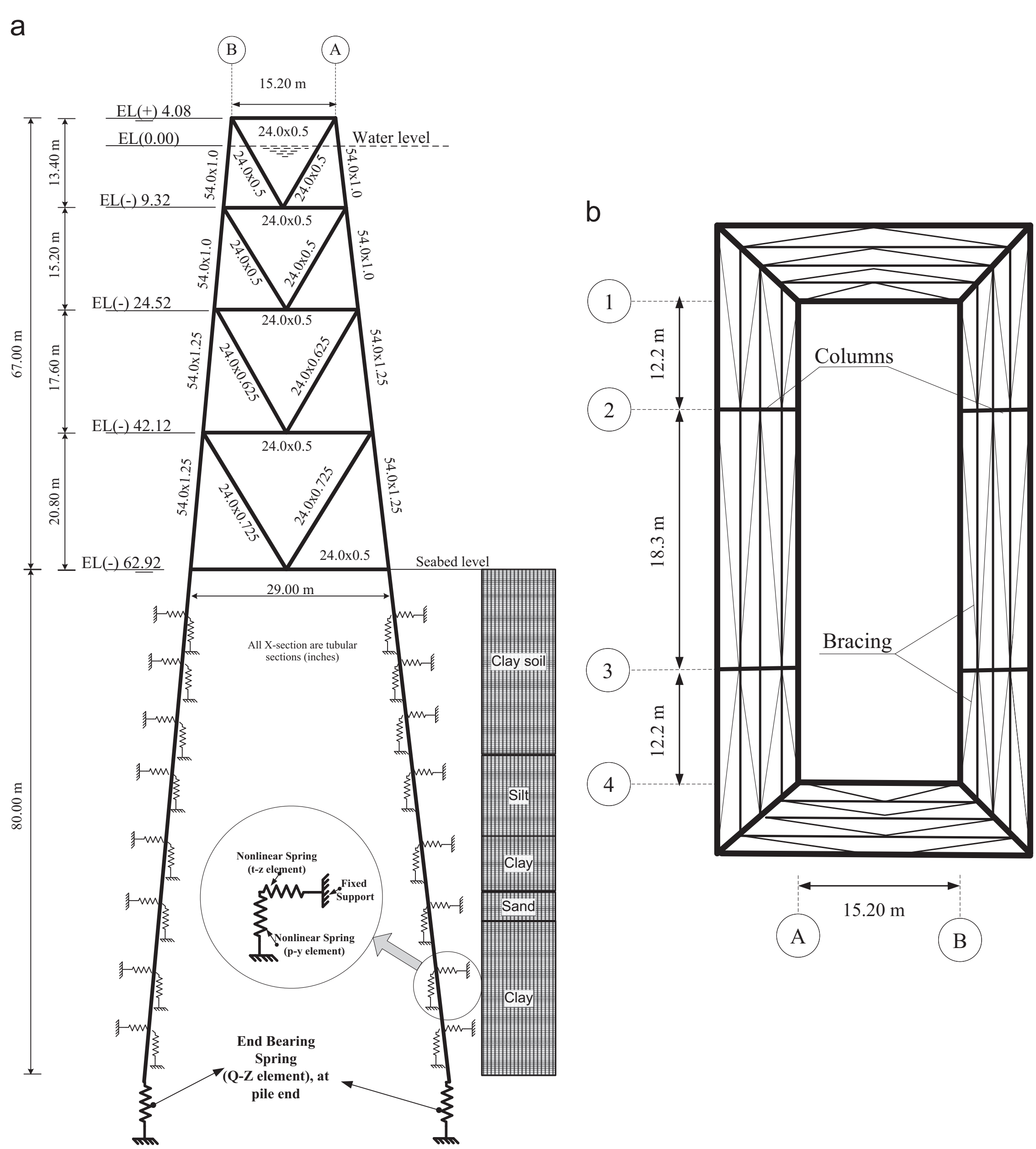

Fig. 5. Jacket structure schematic views, (a) 2-D single frame extracted from the actual platform with the pile soil configuration and (b) plan view of the jacket.

damping ratio of 5% of critical damping is generally used in the analysis of offshore structures (API, American Petroleum Institute Recommended Practice for Planning, 2000), which includes the effect of water–structure interaction and the foundation and structure related energy dissipation effects. In this study the uncertainty in the nonlinear behavior of the model structure is considered by scaling every force and deformation value on the force–deformation relationship by a single random variable (Porter et al., 2002).

5. Modeling of piles

Structural behavior of offshore platform in the non-linear range depends primarily on the pile–soil–structure interaction. In the present study, the Beam on Non-Linear Winkler Foundation (BNWF) model is used (Matlock, 1970), in which parallel nonlinear soil–pile springs are used along the pile penetration length to approximate the interaction between the pile and the surrounding soil. This model simplifies the interaction between the soil and the pile by assuming that the displacement of one spring has no effect on the displacement of other springs. The lateral soil stiffness is modeled using the *p–y* approach. In this approach, for each layer of soil along the depth, a nonlinear relationship is established between the lateral pile displacement (y) which mobilizes the lateral soil reaction (p) per unit length. The procedure of generating *p–y* curves is recommended in API, American Petroleum Institute Recommended Practice for Planning (2000). In the present study, *p–y* curves are based on the actual soil data extracted from the geotechnical report of the platform site (PTTEP International, 2010). In Fig. 7, an example of *p–y* curves for soft clay lateral load–deflection curves extracted from Matlock (1970) is compared with those used in the present study. In the numerical model proposed in this paper, the Multi-Linear Plastic type link element in SAP2000 is used to model the non-linear lateral relation between the soil and the pile. In that link element, the nonlinear link stiffness for the axial degree of freedom is defined according to the *p–y* curve. Then the *p–y* curve is redefined as a force–deformation (*F–D*) relationship where *F* is the total force acting along the tributary length of a pile joint. After that, a lateral link is defined for each joint along each unit pile segment to represent the lateral soil non-linear behavior. Fig. 8 shows the configuration of the proposed model in SAP2000. A multi-linear kinematic plasticity property type is selected for uniaxial deformation from the SAP2000 library to model the hysteresis of the non-gapping soil behavior.

The skin friction and the end bearing between a pile and the surrounding soil produce the soil resistance to the axial movement of the pile. Each of the resistance action is characterized by a

Table 5
The first four modal periods and characteristics of the structure.

Mode	Modal period	Modal participating mass (for each mode) (%)	Modal participating mass (cumulative) (%)
1	1.92	82.1	82.1
2	0.53	7.6	89.7
3	0.42	0.0	89.7
4	0.27	0.6	90.3

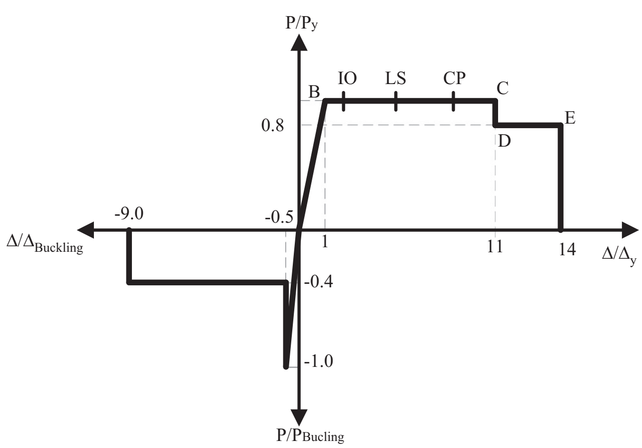

Fig. 6. Back bone curve of the brace plastic hinge hysteresis model based on FEMA-356 for tubular braces.

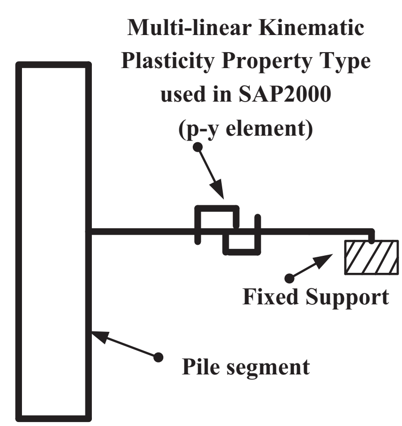

Fig. 8. Configuration of lateral soil stiffness modeled in SAP2000.

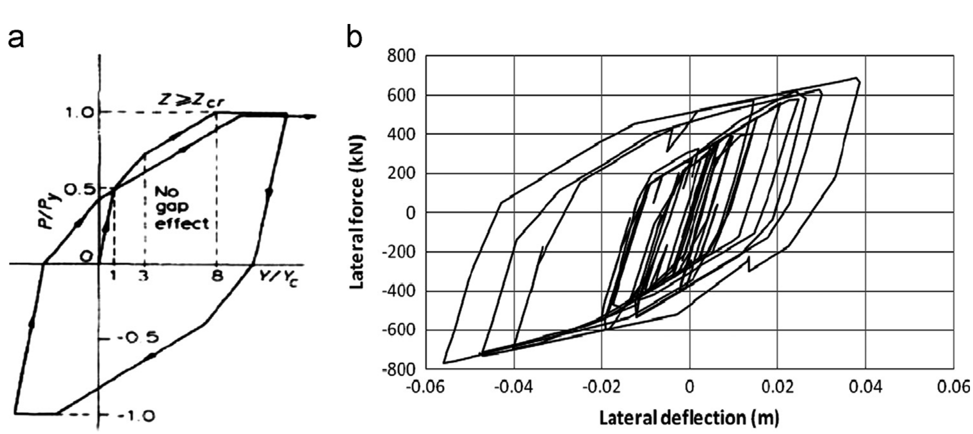

Fig. 7. Lateral load–deflection curves for soft clay; (a) from Matlock, 1970 and (b) model used in present study at pile top (for NGA# 892).

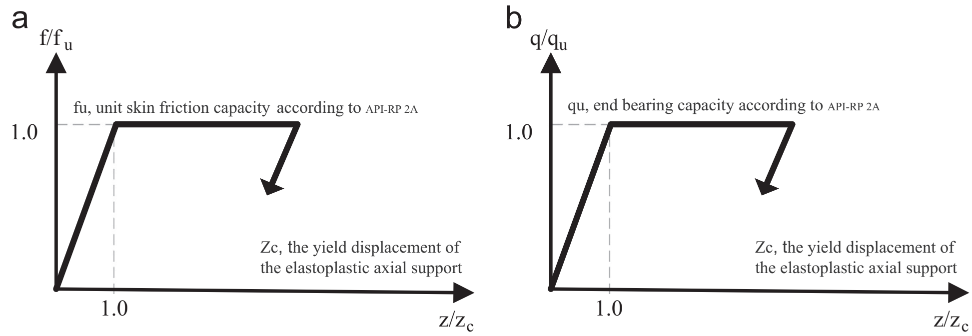

Fig. 9. Axial load–deflection curves for clays and sands (Anagnostopoulos, 1983; Coyle and Reece, 1966). (a) Skin friction and (b) end bearing.

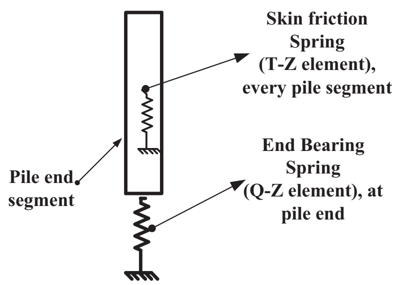

Fig. 10. Schematic illustration of the pile spring model.

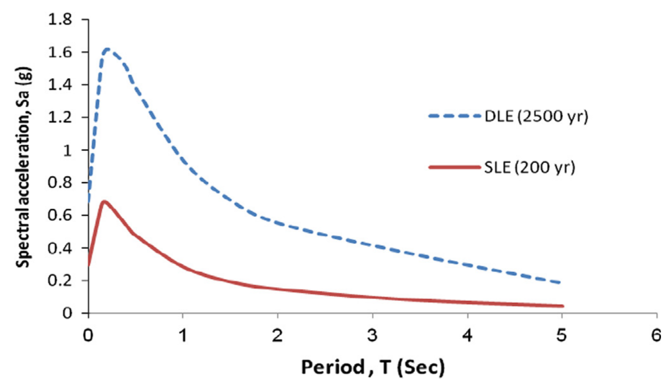

Fig. 11. Response spectra used for strength and ductility level seismic design.

nonlinear force–deformation relationship. Experimental results suggest that these force–deformation characteristics may be adequately represented by the elastic, perfectly-plastic relationship (Anagnostopoulos, 1983; Coyle and Reece, 1966) as shown in Fig. 9.

Frame element is chosen from the library of the SAP2000 to model the behavior of a pile. The diameter of the pile is uniformly 1210 mm and penetrates into 80 m in the soil. In order to simulate the structure–pile–soil interaction through several layers of different soils, the piles are divided along their vertical axis such that within each layer of the soils the portion of the pile is divided into 1.0 m long segments. The relative movement between the pile and soil can be simplified into a number of non-linear vertical springs representing the vertical friction force exerted by the soil on the pile surface. For each pile there is also an end support spring which represents the end-bearing capacity of the pile. Fig. 10 illustrates the arrangement of the vertical and end bearing soil springs. The spring parameters are calculated according to the site investigation and pile testing data (PTTEP International, 2010).

6. Seismic design of the model structure

There are two main requirements stipulated in API, American Petroleum Institute Recommended Practice for Planning (2000) which must be considered in the seismic design of fixed offshore platforms, which are the strength and the ductility requirements. In the present study the response spectra for the strength and the ductility level earthquakes are used in the seismic design of the platform structure. Site specific response spectra for the earthquake with 200 and 2500 year return periods, shown in Fig. 11, obtained from the geotechnical earthquake engineering report (PTTEP International, 2010) are used for strength and ductility based seismic design, respectively. Strength seismic design is required to make a platform adequately sized for strength and stiffness to ensure no significant structural damage during an earthquake shaking which has a reasonable likelihood of not being exceeded during the life of the structure. In the present study, response spectrum analysis method is used in the design of the platform to resist the inertially induced loads produced by the strength level ground motion. The complete quadratic combination (CQC) method is used for combining modal responses. Total of four vibration modes are included to obtain a combined modal mass participation of at least 90 percent of the structure actual mass for an adequate representation of the dynamic response of the 2-D jacket structure. For design load cases, seismic load is combined with other loads such as gravity, buoyancy, and hydrostatic pressure. Ductility requirements are intended to ensure that the platform has sufficient reserve capacity to prevent its collapse during rare intense earthquake motions. Representative set of ground motion records that are characteristic of a rare, intense earthquake at the site are developed from a site-specific seismic hazard study following the provisions of API, American Petroleum Institute Recommended Practice for Planning (2000). The *P*-delta effect of loads acting through lateral deflection of the structure is considered in the analysis. It should be demonstrated that the platform remains stable under the loads imposed by these ground motions. The platform is considered unstable when the deflections are large enough to cause collapse under the influence of gravity loads. API, American Petroleum Institute Recommended Practice for Planning (2000) requires at least three sets of representative earthquake ground motion records for the rare intense earthquake records needed for ductility level check. In the present study, four representative earthquake ground motion time histories shown in Fig. 12 have been used to check the stability of the model structure based on the uniform hazard spectrum (UHS) with 2% probability of exceedance in 50 years (i.e. recurrence interval of 2500 years) developed for the platform site.

7. Selected engineering demand parameters and analysis results

The type and function of the offshore platform are the main factors in selecting the appropriate engineering demand parameters

(EDP). Generally, as a global performance criterion, the peak displacement at the top of the jacket is required to be maintained within a certain limit to safeguard against excessive deck movement. This movement has an important effect on the expensive deck equipments and modules required for operation as in the case of riser platforms or well head platforms. The inter-story drift ratio (IDR) is another commonly accepted criterion for the global level performance of the structures. In offshore platform structures limiting IDR to appropriate values would prevent brace buckling and maintain the functionality of risers and conductor systems. For the local level demand parameter, the formations of plastic hinges are used. The nonlinear behaviors of structural elements are evaluated based on the performance limit states suggested in the FEMA-356, which are immediate occupancy (IO), life safety (LS), and collapse prevention (CP) limit states.

Fig. 13 shows the variation of structural responses such as the maximum top displacement (MTD) and the maximum inter-story drift ratio (MIDR) obtained from the Tornado and the FOSM methods. The two earthquake records (10th and 90th percentile records) shown in Fig. 14 are used for EQ profile variability. These two earthquakes have been selected after sorting the MTD and MIDR of all earthquake records as shown in Table 6. The 50th

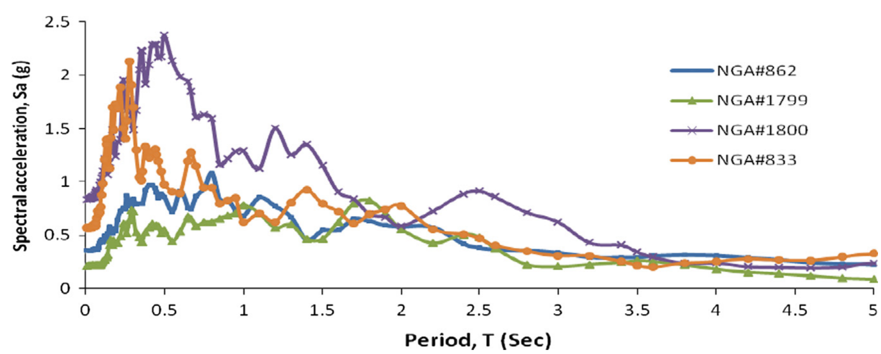

Fig. 12. Response spectra of ductility level earthquakes used to check stability of the model structure.

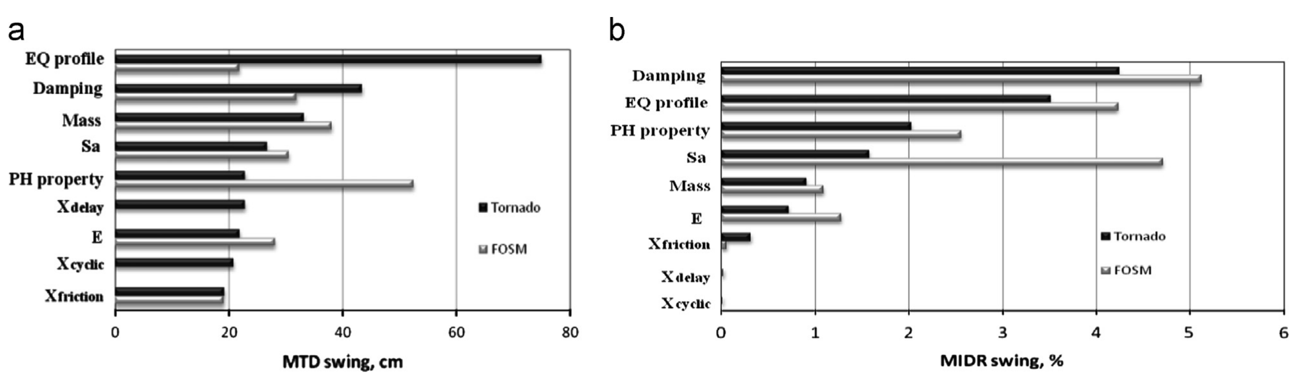

Fig. 13. Sensitivity of the model structure obtained from the Tornado and FOSM methods. (a) Swing of maximum top displacement, (b) Swing of maximum inter-story displacement.

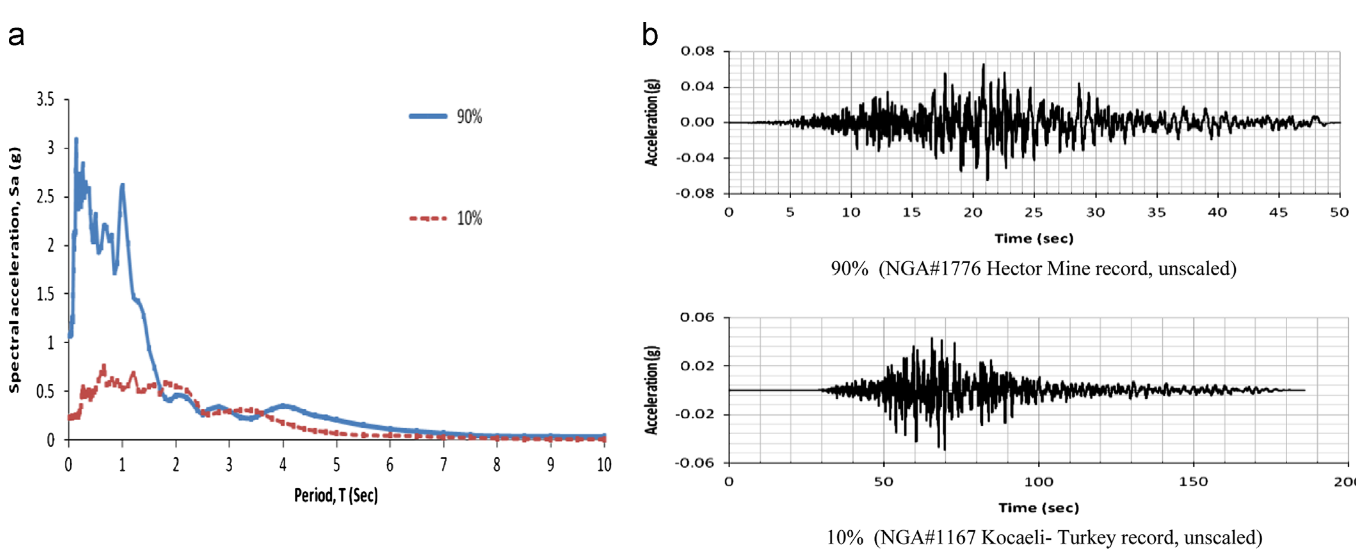

Fig. 14. The 10th and 90th percentile records selected based on the maximum top displacement (MTD) responses for record to record variability. (a) Response spectra and (b) Time history profiles 10% (NGA#1167 Kocaeli – Turkey record, unscaled).

Table 6
Sorted MIDR and MTD values with corresponding ground motion record designations.

MTD (m)				MIDR (%)			
Serial	Record NGA#	MTD (m)	Percentile	Serial	Record NGA#	MIDR (%)	Percentile
1	879	0.06		1	879	0.12	
2	1833	0.17		2	1168	0.38	
3	1168	0.26		3	1167	0.44	
4	1167	0.27	10%	4	1638	0.65	10%
5	1638	0.35		5	853	0.69	
6	853	0.37		6	861	0.71	
7	861	0.37		7	1811	0.86	
8	1811	0.40		8	841	1.1	
9	1148	0.50		9	1833	1.1	
10	1604	0.58		10	1153	1.41	
11	1153	0.59		11	1602	2.28	
12	1805	0.60		12	1148	2.32	
13	833	0.62		13	1604	2.33	
14	886	0.63		14	889	2.35	
15	1800	0.70		15	1776	2.72	
16	1602	0.71		16	892	2.72	
17	1799	0.78		17	833	2.75	
18	841	0.81		18	1636	2.75	
19	892	0.82		19	835	2.78	
20	897	0.84		20	886	2.82	
21	1163	0.85		21	886	2.82	
22	835	0.86		22	1800	3.15	
23	889	0.90		23	897	3.25	
24	12	0.90		24	1799	3.63	
25	862	0.91		25	1163	3.98	
26	1636	0.99		26	862	3.99	
27	1776	1.02	90%	27	1785	4.16	90%
28	1785	1.06		28	1634	4.79	
29	1634	1.07		29	12	4.89	
30	1759	2.04		30	1759	7.63	

percentile record (NGA#892) depicted in Fig. 2 is used to investigate the sensitivity of the other variables using non-linear time history response analysis. A deterministic sensitivity analysis is performed to determine the relative significance of each uncertain variable to EDP uncertainty shown in Fig. 13. In this analysis, it is assumed that the output variable, which is MTD or MIDR, is a known deterministic function (e.g. finite element model (FEM)) of a set of input variables whose probability distribution is described in Tables 1 and 2. For each input variable, the best estimate and two extreme values corresponding to upper and lower bounds, which are 10th and 90th percentiles, of its probability distribution are selected. First, the deterministic function is evaluated to determine the best estimate of the output variable using input variables set to their best estimates (i.e. 50th percentile). Subsequently, for each input variable, the function is evaluated twice using one of the extreme values each time while the other input variables are set to their best estimates (i.e 50th percentile). This process yields two bounding values of the output variable (EDP) for each input variable. The absolute difference of these two values, referred to as the swing, which is illustrated in Fig. 13, is used as an indicator of the "significance" of the given input variable to the output variable. In Fig. 13, the input variables are ranked according to their swings. A larger swing implies a more significant input variable to the uncertainty of the output variable. This applies for all input parameters mentioned in Tables 1 and 2.

It is observed that the EQ profile variability has the greatest impact on the maximum top story displacement (MTD) based on the Tornado method; this appears to be attributable to the inherent record-to-record variability of the ground motion profiles. Among the parameters, the damping ratio has the most significant effect on the variability of the maximum inter-story drift ratio (MIDR) either from Tornado or FOSM method. This is due to the high COV value (0.02/0.05 = 40%) of the critical damping ratio. The effect of the variation of plastic hinge property is relatively small because the structure undergoes little inelastic deformation. Since the MTD and MIDR are the maximum values, they do not occur at the same time in the nonlinear dynamic time history (NLTH) analysis. It also can be observed that the variables such as F_y and $X_{bearing}$ impose little influence on the MTD. The parameters X_{cyclic} and X_{delay} do not affect the responses according to the results of FOSM whereas they have some impact on the MTD based on the Tornado method.

Table 7
Variability coefficients of the displacement responses.

Variable	COV (%)	MIDR		MTD	
		Tornado	FOSM	Tornado	FOSM
Damping ratio	40.0	1.45	1.75	0.51	0.38
EQ profile	–	1.20	1.44	0.88	0.26
PH property	20.0	0.69	0.87	0.27	0.62
S_a	27.2	0.54	1.61	0.31	0.36
Mass	6.0	0.31	0.37	0.39	0.45
E(Young's modulus)	5.0	0.25	0.44	0.26	0.33
$X_{friction}$	26.0	0.11	0.02	0.23	0.23
X_{delay}	7.0	0.01	0.00	0.27	0.00
X_{cyclic}	2.3	0.01	0.00	0.24	0.00
F_y	5.0	0.00	0.00	0.00	0.00
$X_{bearing}$	47.2	0.00	0.00	0.00	0.00

Table 7 shows the variability of the variables obtained from the Tornado diagram method and FOSM method. The variability coefficient represents the ratio of the swing value and the response obtained using the median value of a parameter. It can be observed that the COV of the selected EDP are highly affected by the variability of the ground motion uncertainties and the plastic hinge property as well as the damping ratio. This is attributed to the high COV of those input parameters. Moreover, it is observed that the COVs of the MIDR are much higher when compared to the COVs of the MTD, and that the yield stress F_y and the end bearing force $X_{bearing}$ have little impact on the selected EDPs. The variability coefficient of the effect of time since the pile is driven, X_{cyclic}, is obtained as 24% from the Tornado Diagram method while no variability is observed from the FOSM method.

Fig. 15 shows the swing of the number of plastic hinges formed in the B-IO and C-D deformation states observed at the end of the nonlinear time history analysis using the median record NGA#892-FN. The plastic hinge deformation limit states are based on the acceptance criteria of FEMA-356. These limit states are chosen as a representative of the critical portions of the plastic hinge force–deformation back bone curve shown in Fig. 6. It is observed that the plastic hinge swing in the C-D range are higher than those of the B-IO range for the earthquake profile and the shaft friction $X_{friction}$ parameters, whereas the opposite is true for most of the other parameters. The parameter X_{cyclic} and the damping ratio have the same impact on both limit states. It also can be noticed that the uncertainties in the mass, plastic hinge properties, and the set-up time X_{delay} have the highest impact on the swing of plastic hinge formation in the B-IO range whereas they have small impact on the C-D range of plastic hinge deformation. This means that uncertainty of a parameter may have different impact depending on deformation states.

8. Conclusion

In this paper the effect of uncertainties associated with the design of a fixed type steel offshore platform was investigated

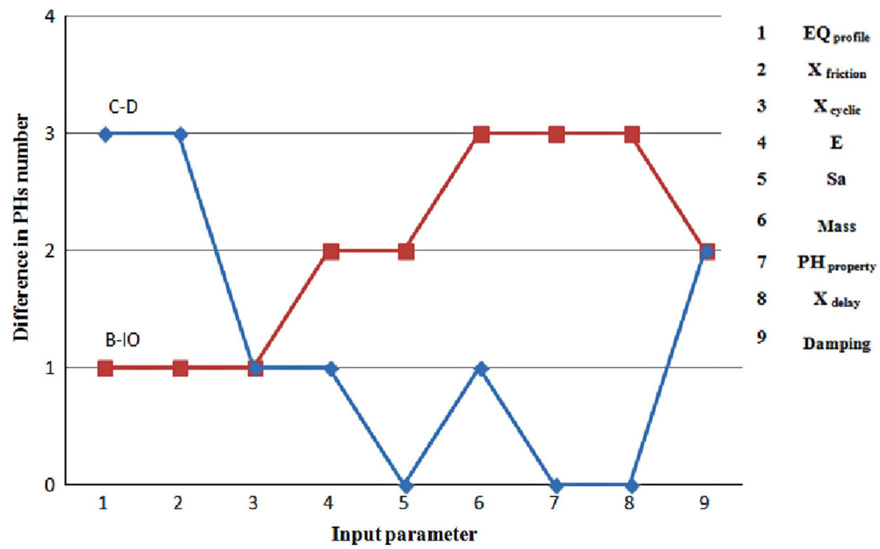

Fig. 15. Swing of number of plastic hinges at the final stage of the analysis.

through FOSM and Tornado diagram methods. According to the analysis results the ground motion uncertainty had a more dominant influence on the selected engineering demand parameters (EDP) compared to the other sources of uncertainty. In comparison the uncertainties associated with the soil–pile uncertainties were found to have a modest effect on the selected EDPs of the fixed type steel offshore platform. Among the uncertainties related to structural modeling, the influences of variables such as damping ratio, mass, and plastic hinge property were somewhat significant. The detailed findings of this paper are summarized as follows:

(i) The earthquake profile variability had the most significant impact on the maximum top displacement (MTD), while it has the second highest impact on the maximum inter-story drift ratio (MIDR). The effect of the spectral acceleration S_a was viewed as moderate on both MTD and MIDR.

(ii) Among the structural uncertain parameters the variability in damping ratio had the most significant impact on both the engineering demand parameters (EDP), whereas the mass, elastic modulus, and the plastic hinge properties showed relatively moderate effects.

(iii) $X_{friction}$ proved to be the most important uncertain parameter among the soil–pile modeling parameters with higher impact on MTD compared to that on MIDR. The effect of X_{delay} and X_{cyclic} is observed by the Tornado method only with the tendency to affect MTD more than MIDR, whereas $X_{bearing}$ found to have almost no influence on the selected EDPs.

(iv) The sensitivity of design parameters varied depending on the deformation stages of model structures. Mass, HP property, and X_{delay} had the highest impact on the "B-IO" range of displacement while the influence of the earthquake profile and $X_{friction}$ were significant in the "C-D" range.

Acknowledgments

This research was supported by a grant (13AUDP-B066083-01) from Architecture & Urban Development Research Program funded by Ministry of Land, Infrastructure and Transport of Korean government, and by the Samsung Engineering Co., Ltd.

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