Incremental Dynamic Analysis of Jacket Type Offshore Platforms Considering Soil-Pile Interaction

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ABSTRACT :

The exploitation for offshore oil has become increasingly widespread in the past decades. Steel offshore platforms are commonly constructed to accommodate operation facilities and to withstand environmental and accidental loads during oil exploitation operation. Particular attention is being paid to earthquake loads in seismic active areas because it directly influences the performance of the offshore structure installations. Incremental Dynamic Analysis (IDA) is a powerful tool to assess the capacity of a structure subjected to seismic loads. In this paper, IDA analysis is performed on a newly designed and installed offshore platform through the use of nonlinear finite element program. Pile-soil interaction and post-buckling of lateral load resisting elements are considered and relevant demand parameters are introduced. Nonlinear pile model is used to capture the failure of piles so that the capacity and demand of the structure can be estimated more accurately. It is concluded that using different demand parameters may change the summarized IDA curves especially in highly nonlinear responses of the structure.

KEYWORDS: Incremental Dynamic Analysis (IDA), Engineering Demand Parameters (EDP), Jacket, Pile nonlinearity, Soil-Pile-Structure Interactions

1. Introduction

The Increasing demand for energy has led human beings to search for oil and gas beyond its landlocked properties. Excavation for oil in offshore areas has begun for several years. This industry relies on offshore structures as part of their existence. Steel platforms are one of the most common types of structural systems currently used for oil exploitation purposes. These structures are generally designed to resist environmental loads namely, functional loads and loads due to waves, currents, wind as well as earthquake excitations. Particular attention is being paid to ground motion induced loads in seismic active areas since these loads may cause damage and subsequent collapse of a structure. As such, the consequences of these environmental effects should be accounted for in design of an offshore installation. Incremental Dynamic Analysis [18] is a powerful tool to assess the global and local capacity of structures. This method of analysis may provide several insights regarding dynamic characteristics of a structure as well as useful inputs for applications of performance-based evaluation. The latter has been widely used by engineers to manage the cost of construction as well as maintaining the safety of structures. It has formed applications not only in buildings but also in any type of expensive and important structure. IDA was well documented and introduced by Vamvatsikos and Cornell (2005) [18] but the concept of seismic load scaling had been formerly used by several authors such as Bertero (1977) [5], Luco and Cornell (1998) [10], Mehanny and Deierlin (2000) [14], Nassar and Krawinkler (1998) [15] and Zargar et al. (2008) [20]. They used this concept to assess the performance of structural frames in buildings but its application in offshore platforms has not yet been comprehensively investigated.

The behavior of pile foundations under earthquake loading is an important factor affecting the performance of many essential structures. Analysis and design procedures have been developed for evaluating pile behavior under earthquake loading. Dynamic *p-y* analyses have a long history of development and application to seismic and offshore problems [e.g., Matlock et al. (1978) [11, 12], Kagawa and Kraft (1980) [9], and Nogami et al. (1992) [16]]. It can be performed using a number of different computer codes. Various approaches have been developed for the dynamic response analysis of piles. One such method which will be used throughout this paper is the Beam on Nonlinear Winkler Foundation (BNWF) model, where the soil-pile interaction is approximated using parallel nonlinear soil-pile (p-y) springs (Matlock 1978) [11, 12]. Predicting the behavior of pile foundations under earthquake loading involves consideration of earthquake characteristics, free field site response, soil profile characteristics, superstructure response, and soil-pile-superstructure interaction. A common design approach in the world today is to avoid inelastic behavior of piles and their connections below the ground surface, where damage would be difficult to detect or to repair.

In this study, a newly designed and installed jacket type offshore platform is modeled using nonlinear structural and geotechnical program Opensees [13]. Element verification and formulation is concisely reviewed in order to prove the suitability and reliability of software for dynamic analyses. Soil-Pile interaction and nonlinear pile model as well as post-buckling of members have been considered to simulate the realistic behavior of an offshore platform as accurately as possible. Two different IMs and EDPs are examined and an appropriate EDP for jacket type offshore platform is addressed.

2. Element Verification

In Order to perform IDA analyses, analytical models should be composed of robust and well-tested elements to avoid any unrealistic and deficient responses. Typically, Jacket Type Offshore Platforms are made of struts, portals and piles for which uniaxial elements may be used to model the behavior of individual members. Interface elements such as soil-pile- Interaction elements also need to be used so that all components of a structure are modeled analytically.

2.1 Struts

For modeling of struts, beam-column element is used. Nonlinear beam-column element models which have been widely used to model inelastic post-buckling and cyclic behavior of steel braces are classified as finite element, phenomenological and physical theory models. Physical theory

models are based on simplified hysteretic rules and consequently they are computationally less expensive, whereas their finite element counterparts are more versatile and sophisticated.
For modeling the post-buckling and cyclic behavior of braces, beam-column element formulations should account for material and geometric nonlinearity as well, particularly when member undergoes large inelastic displacements and deformations.
Using the finite element program Opensees [13], one can find that one of the most efficient elements [7] is the nonlinear force based beam-column element. This element is developed by De Souza (2000) [7] and when used with corotational transformation, is capable of accounting for large displacements (i.e. post-buckling behavior is fully modeled). At the same time, material nonlinearity is also considered through the use of fiber discretization of the cross section. A snap-through model is used for brace representation with different end conditions. The model consists of two force based beam-column element with 1/1000 of its length as snap. Five integration points are defined along the element to account for distributed plasticity. This kind of modeling has proved to be a good representation of a real and imperfect brace which is used in structures [2].

2.2 Portals

Jacket legs usually behave as portals whose behavior can be simulated by a cantilever beam-column under a constant axial load and varying lateral load. This is due to relatively small bending moment at the mid-height of the leg segment between any two adjacent horizontal bracing levels. In this work, portals are modeled by means of one nonlinear force beam-column element in which five integration points are defined to account for distributed plasticity.

2.3 Nonlinear p-y Elements

Nonlinear *p-y* behavior was modeled using the element described in Boulanger et al. (2004) [6], which accounts for gapping and radiation damping. The *p-y* parameters for the soft clay were based on Matlock's (1970) [11, 12] recommendations, and the *p-y* parameters for the underlying sand were based on [American Petroleum Institute (API) 1993 [1] recommendations. One P-Y element was tested with the finite element program Opensees [13] and the resulting *p-y* curves match Matlock's within a few percent, over the entire range of *y* as shown in figure 1. Also a sketch of *p-y* element is shown in this figure representing a nonlinear spring and a damper as well as gap component. For this study, the input parameters p_{ult} and y_{50} were also based upon Matlock's (1970) [11,12] equations which are as follows:

$$P_{ult} = C_u B N_p \tag{2.1}$$

$$N_p = \left(3 + \frac{\gamma' x}{C_u} + \frac{Jx}{B}\right) \leq 9 \tag{2.2}$$

$$y_{50} = 2.5 B \varepsilon_{50} \tag{2.3}$$

Where B = pile diameter; N_p = lateral bearing capacity factor; γ' = average buoyant unit weight; x = depth; C_u = undrained shear strength; and ε_{50} = strain corresponding to a stress of 50% of the ultimate stress in a laboratory stress-strain curve, and J was taken according to Matlock's recommendations for soft clay. The gapping behavior includes a residual resistance that may be thought of as a drag force on the sides of the pile as it moves within the gap. This residual resistance is specified as a ratio of ultimate resistance p_{ult} by a parameter C_d. This parameter is assumed to be 0.3 for clay according to the centrifuge tests done by Wilson et al. (1998) [19]. Incremental Dynamic Analysis on single piles using the p-y, t-z and q-z materials where carried out by Assareh et al. (2008) [3].

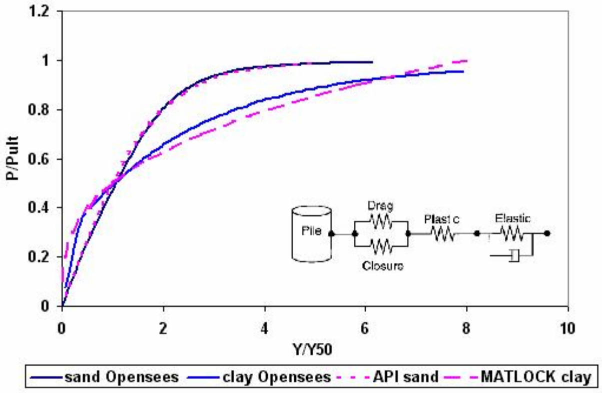

Figure 1 p-y behavior in different models

2.3 Nonlinear t-z and q-z Elements

Nonlinear *t-z* and *q-z* elements for skin friction on the piles were modeled as elastic and plastic components in series as described by Boulanger et al. (2004) [6]. The ultimate skin friction resistance of the *t-z* elements in the clay and sand was calculated using the method presented in API [1]. For the sand, it was calculated using the shaft friction $f = KP_0 \tan\delta$, where k is the coefficient of lateral earth pressure, P_0 is the effective overburden pressure at the point and δ is the friction angle between the soil and pile wall. For the clay layers the t-z elements are modeled according to the API recommendations. For cohesive soils shaft friction can be represented with $f = \alpha c$, where α is equal to unity and c is the undrained shear strength of the soil at the point. Nonlinear *q-z* elements for the pile tip resistance were also modeled as elastic, plastic, and gap components in series as stated by Boulanger et al. (2004) [6]. The *q-z* element are modeled using the recommendations of API [1] for clay (pile tip rests in stiff clay). The ultimate bearing capacity was calculated as $q = 9cA$ where A is the area of the pile cross-section at the tip. Application of p-y, t-z, and q-z elements are depicted in figure 2.

3. Case Study

3.1. Sample well-head platform

Well-head platforms, production platforms as well as living quarters are the most frequent types of platforms to be constructed for exploitation of oil purposes. A newly designed and installed well-head platform in Persian Gulf has been selected to be modeled analytically as a case study. Figure 3 shows this four legged platform along with analytical model built by Opensees. Only the major structural components were included within the models and the contribution of appurtenances and conductors to platforms' stiffness and strength were neglected. In order to capture the buckling of braces, 2 force-based nonlinear beam-column elements have been used as mentioned previously. Table 1 shows material and geometrical properties of the platform.
Detection of damage which is likely to occur during IDA analyses is an important objective of this study. Hence, in addition to the jacket members, each pile has been modeled in such a way that every element is one meter long. These types of elements were chosen to capture nonlinear response in piles which has not been studied in detail in the past.

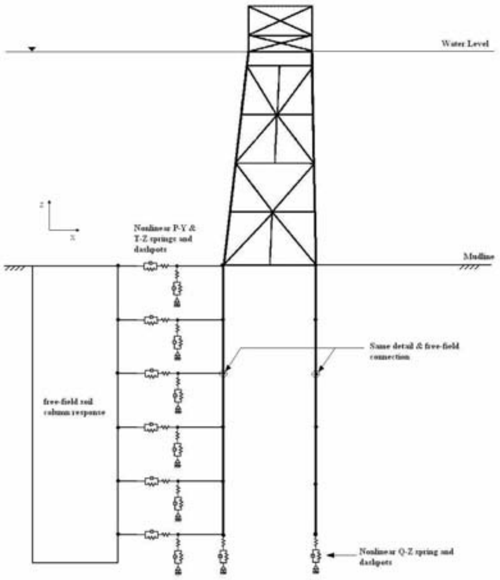

Figure 2 schematic drawing of the application of p-y, t-z, and q-z elements on jacket's piles

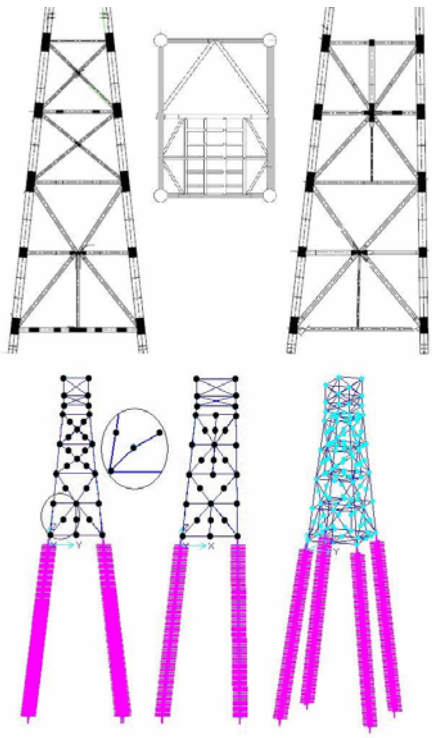

Figure 3 Sample well-head platform shown in different directions and modeled in OPENSEES

Each pile node below the ground surface was connected to two nonlinear *p-y* elements (described earlier) in two directions. The t-z elements were also placed on the nodes between the piles in every meter to apply the skin friction of the piles. As for the end bearing resistance, a q-z element was used under each pile. These springs were modeled as the zero-length elements that generally share one node. Displacement time histories from the free-field site response analyses should be input to the fixed ends of the *p-y* elements but they should be modified considering the soil properties a priori. In this paper the response of the soil profile was analyzed using CYCLIC1D (a one dimensional nonlinear finite element program) [8] which is developed by Yang and Elgamal (2001) [8]. Soil properties are given to the program based on the geotechnical data as mentioned in Table 2.

Table 1 Material and geometrical properties of the platform

Geometrical and environmental properties		Material properties		
Water depth	60.3 m	Elastic modulus	16mm<thick.<40mm	16mm≤thick.
Jacket height	69.3 m		210e9 N/m^2	210e9 N/m^2
Jacket dimension	$13\times 20\mathrm{m}^2$	Yield stress	345e6 N/m^2	355e6 N/m^2
Total No. of jacket legs	4			
Total No. of jacket piles	4 Ungrouted	Tangent modulus	441e7 N/m^2	483e7N/m^2
Pile dimensions	$1524\times 60\mathrm{mm}^2$			
Pile penetration	88 m			

Table 2 soil layers and properties

Layer No.	Depth (m)		Description	γ' (KN/m^3)	C_u (KPa)	φ (deg)	N_q	ε_{50} (%)
	From	To						
1	0	0.2	Medium dense sand	6.5	-	25	12	
2	0.2	2	Firm clay	7.5	45	-	-	3.5
3	2	2.5	Medium dense sand	8	-	30	20	-
4	2.5	4.5	Firm to stiff silty clay	8	50	-	-	2
5	4.5	7.85	Medium dense sand	8	-	30	20	-
6	7.85	12	Soft to firm clay	7.5	35-50	-	-	2
7	12	29.6	Stiff clay	7.5	100-120	-	-	2
8	29.6	31.5	Strong limestone	8	-	30	20	-
9	31.5	52.7	Very stiff to hard clay	9.5	175-225	-	-	2
10	52.7	57.5	Medium dense sand	8	-	30	20	-
11	57.5	68.7	Hard clay	9	300-350	-	-	2
12	68.7	69.5	Strong limestone	8	-	30	20	-
13	69.5	88.8	Hard to very hard clay	9	350-450	-	-	1.5
14	88.8	89.5	Strong Limestone	8	-	30	20	-
15	89.5	103.6	Very hard clay	9	450	-	-	1.5
16	103.6	111	Very hard clay	9	450	-	-	1.5

3.2. IDA analysis

Obviously, the input and output of an IDA analysis are Intensity Measures (IM) and Engineering Demand Parameters (EDP) respectively. As nonlinear dynamic analysis becomes a more frequently used procedure for evaluating the demand on a structure due to earthquakes, it is increasingly important to understand which properties of a recorded ground motion are most strongly related to the response caused in the structure [5]. Consequently, a value should be defined to quantify the effect of a record on a structure which is IM. Several IMs have been defined since its introduction among which peak ground acceleration (PGA) and spectral acceleration at the first-mode period of vibration ($S_a(T_1,5\%)$) as well as variety of vector-valued IMs, may be mentioned. Shome et al. (1998) [17] found $S_a(T_1,5\%)$ to be more effective due to its structure-specific characteristic, compared to PGA which is totally site-specific. Hence in this study $S_a(T_1,5\%)$ is used as the preferred IM.

On the other hand, EDP which is the response of the structure to the seismic loading should be selected appropriately considering the characteristics of the structural system which is being studied. In case of buildings, maximum interstory drift may be selected knowing the fact that foundation rotations are not severe. Moreover, by using maximum interstory drift, one can avoid non-structural damage to the building by setting appropriate limit states and preventing the exceedance of drift from certain values. On the other hand, for offshore platforms, some other aspects should be taken into account and EDP should be selected depending primarily upon the type of the platform. In well-head platforms, deck equipments are very expensive in comparison with jacket structure individually. Therefore in these platforms, overall drift should not exceed a certain limit value and should be monitored through the use of suitable choice of EDP. Additionally, candidate EDP should account for both brace buckling as well as pile failure. The former can be taken into account by peak interstory drift whilst the latter requires more investigations. Based on the above, three types of EDP may be proposed. (1) Interstory drift by which buckling of braces in any type of structure can be captured. (2) Peak drift from mud line up to deck level which is suggested in this paper as a suitable parameter to account for both brace buckling as well as pile failure in most geotechnical conditions and finally (3) Overall drift from the lower level of the pile under the ground up to the deck level. The latter does not appear to be so representative since different forms of deformation of pile may take place when subjected to external actions.

As shown in Table 2, for this platform 12 meters soil profile beneath the mud line constitutes much weaker strength in comparison with other layers since it mainly consists of stiff clay with undrained shear strength of approximately 120-400 KPa. Later, it will be shown that the displacements in the top 20 meters of the pile are much more considerable than the other elements of the pile (figure 4.a). As displacements of pile gets greater, then strains and stresses become more significant and cause the pile to fail gradually during loading process. As a result, two limit ideal situations can be considered as shown in Figure 4.b. Fixed-ended platform represents unloaded condition whereas pin-ended platform represents the situation in which lateral displacements at the mud line are excessive so that piles yield totally and form plastic hinges. Obviously an intermediate situation is always encountered in practice when earthquake excitations are applied to the structure. Particularly in IDA analysis, as records are scaled to reach the structural instability and occurrence of plastic hinge, Which is quite probable. It is obvious that pin-ended frame shows greater drift compared to fixed-ended one. It can be concluded that if peak drift (as defined previously) is introduced as an EDP, both buckling of braces and failure of piles can be captured simultaneously.

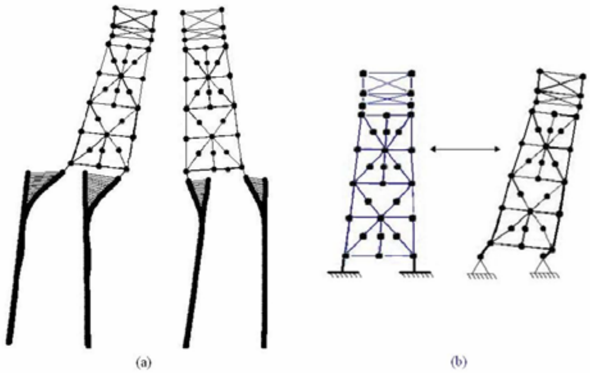

Figure 4 a) displacements of the pile and Jacket during the dynamic analysis b) The two limit situations considered for the model (Fixed-ended platform and pin-ended platform)

3.3. Analysis results

In order to perform IDA a suite of twenty records representing a scenario earthquake was selected (Table 3). Theses records had been originally used by Vamvatsikos and Cornell (2002) [18] and they belong to a group of relatively large magnitude and moderate distance records. Each record was scaled to cover the entire range of structural response and applied to the analytical model using $S_a(T_1,5\%)$ as IM and structural response was recorded via both maximum interstory drift ratio and peak drift as described previously. Responses of the structure to record 14 and 1 as two sample earthquakes containing quite different frequency content have been individually selected to be studied in this section to characterize the behavior of different parts of the structure during single recorded IDA.

Closer investigation for piles proves that lateral displacements at the mud line are as excessive due to weak strength of soil, exactly beneath the seabed as mentioned in previous discussions. This can be demonstrated by a similar story-like single recorded IDA as shown in figure 5.a and b. As can be observed in figure 5.a, pile has been divided into 22 meters pieces to represent 4 virtual stories and has been studied individually during an IDA analysis. Results show much greater drifts at mud line level in comparison with other stories. Figure 5.b shows the pile profile from the tip, when subjected to different seismic inputs, all proving the excessive displacements occurring at the mud line.

Table 3 The set of twenty ground motion records used

No	Event	Station	$\phi^{\circ\,1}$	$Soil^2$	M^3	$R(Km)^4$	$PGA\ (g)$
1	Loma Prieta, 1989	Agnews State Hospital	090	C,D	6.9	28.2	0.159
2	Imperial Valley, 1979	Plaster City	135	C,D	6.5	31.7	0.057
3	Loma Prieta, 1989	Hollister Diff. Array	255	-,D	6.9	25.8	0.279
4	Loma Prieta, 1989	Anderson Dam Downstream	270	B,D	6.9	21.4	0.244
5	Loma Prieta, 1989	Coyote Lake Dam Downstream	285	B,D	6.9	22.3	0.179
6	Imperial Valley, 1979	Cucapah	085	C,D	6.5	23.6	0.309
7	Loma Prieta, 1989	Sunnyvale Colton Ave	270	C,D	6.9	28.8	0.207
8	Imperial Valley, 1979	El Centro Array #13	140	C,D	6.5	21.9	0.117
9	Imperial Valley, 1979	Westmoreland Fire Station	090	C,D	6.5	15.1	0.074
10	Loma Prieta, 1989	Hollister South & Pine	000	-,D	6.9	28.8	0.371
11	Loma Prieta, 1989	Sunnyvale Colton Ave	360	C,D	6.9	28.8	0.209
12	Superstition Hills, 1987	Wildlife Liquefaction Array	090	C,D	6.7	24.4	0.180
13	Imperial Valley, 1979	Chihuahua	282	C,D	6.5	28.7	0.254
14	Imperial Valley, 1979	El Centro Array #13	230	C,D	6.5	21.9	0.139
15	Imperial Valley, 1979	Westmoreland Fire Station	180	C,D	6.5	15.1	0.110
16	Loma Prieta, 1989	WAHO	000	-,D	6.9	16.9	0.370
17	Superstition Hills, 1987	Wildlife Liquefaction Array	360	C,D	6.7	24.4	0.200
18	Imperial Valley, 1979	Plaster City	045	C,D	6.5	31.7	0.042
19	Loma Prieta, 1989	Hollister Diff. Array	165	-,D	6.9	25.8	0.269
20	Loma Prieta, 1989	WAHO	090	-,D	6.9	16.9	0.638

[1]Component [2]USGS, Geomatrix soil class [3]Moment Magnitude [4]Closest distance to fault rupture

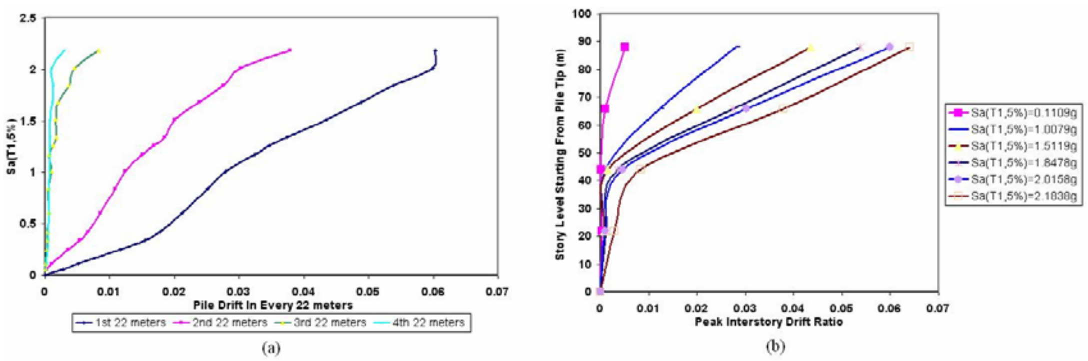

Figure 5 a) Pile drift in every 22 meters Vs. Sa(T1,5%) for the four virtual stories b) Peak interstory drift ratio from pile tip to highest level of the jacket

These displacements induce strains and stresses in pile elements as shown in Figure 6 a and b in which two IDA curves have been plotted in which EDP has been chosen to be normalized stress of the cross section computed from outer fiber. As can be seen, when the level of seismic input arises, pile elements beneath the mud line tend to fail and reach the yield strength. This phenomenon is also seen in lower levels of pile but the amount is not as significant as first few meters beneath the seabed. It can be concluded that the peak drift as an EDP is able to capture nonlinearities induced in piles as described before especially near the flat-lining of IDA curves where pile yielding occurs.

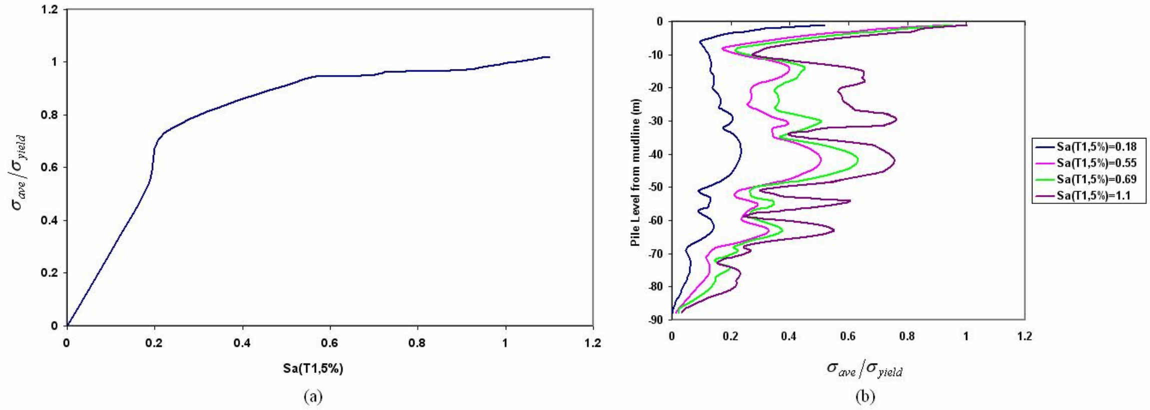

Figure 6 a) A single IDA curve showing the yield strength of the pile b) Yield strength Vs. height of structure for different Sa(T1,5%)

After applying all records on the structure, multi-recorded IDA can be obtained and summarized accordingly. Results of summarized IDA for two different EDPs have been compared to study the differences between them. 16%, 50% and 84% percentiles [18] have been calculated using both peak drift as well as interstory drift as EDP (figure 7). Obviously no apparent difference is seen when the intensity of seismic input is low but as IM arises, differences appears to be significant. In previous section, it was discussed that in the vicinity of $S_a(T_1, 5\%)$ equal to 1, piles reach their yield stress and loose their stiffness rapidly which cause higher drifts. These drifts when incorporated into overall drift of the structure, show greater responses and cause instability sooner. This deterioration of strength is not seen when EDP is chosen to be maximum interstory drift ratio.

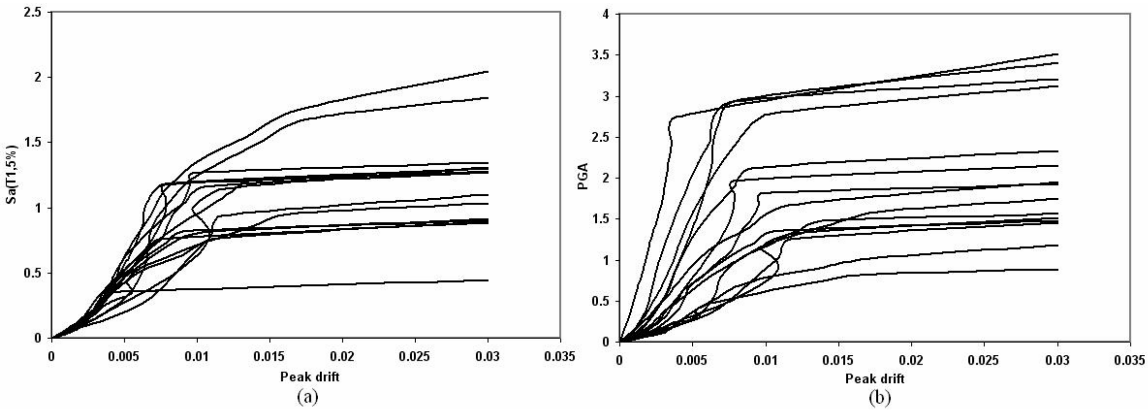

Figure 7 The 20 IDA curves analyzed with a) using the S_a (T1, 5%) Intensity Measure b) using the PGA Intensity Measure

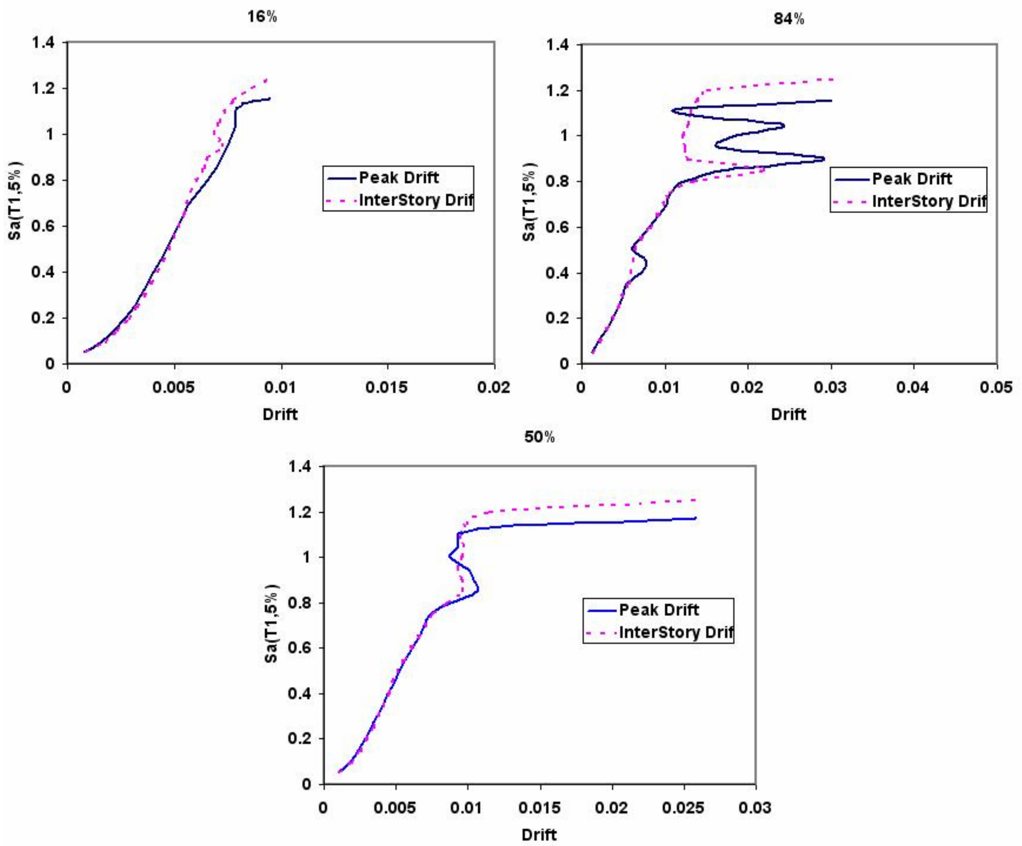

Figure 8 16%, 50% and 84% fractiles for the IDA curves with different Intensity Measures

4. Conclusion

In this paper, an analytical model of an offshore structure was used to assess the performance of an offshore structures using Incremental Dynamic Analysis (IDA). Verified element models were used to obtain the best simulation of the real behavior. Moreover, analysis steps were accurately organized so that unwanted numerical instability problems are avoided. Soil-Pile interaction and nonlinear pile elements were used to account for realistic behavior of foundations. Using nonlinear pile model, peak drift as a suitable engineering demand parameter was compared with the conventional demand parameter used currently in buildings which is maximum interstory drift ratio. It was shown that, peak drift can be utilized to account for both interstory drift as well as pile nonlinearity. Setting appropriate limit states to establish a probabilistic database in order to assess the performance of the structure needs further investigations.

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