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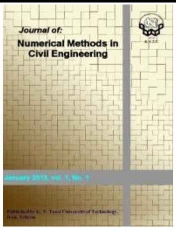

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Sufficiency assessments of ground motion intensity measures employing kullback-leibler theory (applied for typical south pars offshore platforms)

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Abstract:

The potential ingrained uncertainty in ground motion records may significantly influence the structural seismic risk assessment in performance-based earthquake engineering (PBEE). One of the basic components of the socio-economic method of PBEE design is probabilistic seismic demand model (PSDM). The level of uncertainty in PSDM, depends greatly on the selected seismic intensity measure (IM), while these models are traditionally conditioned on a single IM. Among various terms utilized in optimal IM selection, this study particularly aims to bring the "sufficiency" assessment procedures into focus. However, the IM efficiency evaluations have also been considered. The sufficiency of IM is gauged by the extent to which the residual demand measure values are statistically independent of ground motion magnitude (M_w) and distance (R), regressing of IM. The objective of this study is to introduce a recently emerged quantitative procedure by employing relative sufficiency measure (RSM) on the basis of Kullback-Leibler divergence concepts to indicate the superiority of one IM relative to another in the representation of ground motion uncertainty. Besides, the traditional methods of sufficiency evaluation are also discussed. To this end, a three-dimensional finite element model of typical South Pars fixed pile-founded offshore platforms has been built. Several IM candidates are classified and compared in terms of the expected difference in the information they provide for predicting a wide range of structural response parameters. It can be deduced that the most informative of the fourteen considered IMs are among velocity-related ones. The results also demonstrate the absolute necessity of the RSM in optimal IM ranking.

1. Introduction

Highlighted as one of the world's most strategic components, offshore platforms have been the matter of concern. Besides, their failure detrimental financial and environmental effects, made these infrastructures and their seismic assessments pivotal. On the other hand, due to the presence of more than 160 offshore platforms in the Persian Gulf, employing mechanisms to mitigate the structural responses and increasing their lifetime against seismic-induced vibrations, is inevitable.

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Emerging as a significant challenge, seismic risk evaluation is often rooted in probabilistic frameworks, which may account for uncertainties in framework components ranging from the seismic hazard to structural response to consequence assessment, among others.

In this regard, and in the context of Performance-Based Earthquake Engineering (PBEE), the seismic risk for a structure can be expressed in terms of the mean annual frequency (MAF) of exceeding a specified limit state [1,2].

The structural seismic demands are required to be estimated accurately in PBEE. However, the uncertainties in seismic responses caused by uncertainties associated with the input parameters (based on the structural model and/or ground motion intensity measures) are the factors that decrease this accuracy. Probabilistic seismic demand model (PSDM), as one of PBEE basic components, is formulated according to the relation between ground motion intensity measures (IMs) and engineering demand parameters (EDPs) resulting from Probabilistic seismic demand analysis (PSDA). Probabilistic seismic demand analysis (PSDA) of nonlinear structures was performed by Shome [3] and Shome and Cornell [4]. Accomplished by the selection of an appropriate or optimal IM, upon which the demand model is conditioned, tracking and reducing the uncertainty associated with the PSDM is an essential task. Based on the importance of appropriate IM selection, several researchers have explored this issue, and a range of different IMs have been evaluated and adopted for PSDA of buildings and bridges.

The fact that the proposed criteria for optimal IM evaluations is expressed in terms of the information provided for predicting the response quantities (involved in the performance objectives), seems quite logical. Luco and Cornell have proposed sufficiency as well as efficiency, as one of the criteria for assessing the superiority of an IM for representing the dominant features of ground shaking [5]. While efficiency indicates the dispersion in regression analysis of IM and EDPs, a sufficient IM has been defined as one that renders the structural response conditional on this IM to be independent of other ground motion characteristics such as magnitude and distance. The sufficiency condition which they proposed, is an absolute term. Twenty-three different potential IMs for the PSDMs' development of multi-frame highway bridges, have been identified and compared in a study by Mackie and Stojadinović [6]. Among nineteen IMs evaluated by Bradley et al., Housner spectrum intensity, (*HI*) [8], exhibited the characteristics of an optimal IM for the seismic response prediction of pile foundations in liquefiable grounds [7]. Shafieezadeh investigations for pile-supported wharf structures considering the effect of liquefaction, resulted in selection of peak ground velocity (*PGV*) as the optimal IM [9]. Amirabadi et al. proposed $S_a(T1, 5\%)$ and S_{de} as the optimal IMs among those studied for developing optimal PSDMs of pile-supported wharf structures with batter piles, not considering the effect of liquefaction [10]. 38 IM candidates composed of 32 scalar and 6 vector-valued IMs, have been evaluated for selecting suitable IMs for two typical 4-story and 6-story existing RC moment resisting frames by Ebrahimian et al. [11]. Wang et al., comprehensively considered assessment of 26 IMs for extended pile-shaft-supported bridges in liquefied and laterally spreading grounds [12]. It should be noticed that, by using the concept of relative entropy [13] from information theory, Jalayer et al. [14] introduced a measure, called relative sufficiency measure (*RSM*) which has been also employed by Ebrahimian et al. [11] and Wang et al. [12].

For a scalar or low-dimensional vector IM, it is interesting to examine sufficiency in a relative sense; i.e., to investigate whether one IM is more sufficient (more informative) than another for predicting a structural response parameter. The Kullback-Leibler divergence, also called the relative entropy or cross entropy, is expressed in bits of information and has been used in earthquake engineering applications to compare the relative sufficiency of alternative IMs in predicting structural responses. Information theory concepts can be employed to measure the predominance of one IM relative to another for representing ground motion uncertainty [15]. The (Shannon) entropy of an uncertain-valued variable is a measure of the amount of uncertainty in the value of that variable [16-18]. More specifically, it is the measure of the missing information that is required (on average) to identify the value of the uncertain variable. Introducing in the present study, on the basis of the application of entropy and the related concept of Kullback-Leibler divergence (relative entropy), a simple quantitative measure called the relative sufficiency measure (*RSM*), is used for comparing the suitability of alternative IMs. *RSM* declares (on average) how much more information about the designated structural response parameter (EDP) one IM gives relative to another. This work also presents a case study using the relative sufficiency measure besides other IM sufficiency evaluation methods to compare the predominance of alternative IMs for predicting the peak/residual structural responses in typical South Pars fixed pile-founded offshore platforms located in the Persian Gulf.

For the objectives of this study, a 3D finite element model of the platform has been made, considering the previously conducted studies in the offshore platform seismic assessments [19,20] and [40,41]. 80 ground motion records have been selected. Among classified IMs, fourteen candidates which triggered to appropriate performance in prediction of six EDPs (local, intermediate and global) in terms of efficiency have been chosen and the absolute and relative sufficiency of these IM-EDP pairs is evaluated. The overview of the study is illustrated in Figure 1.

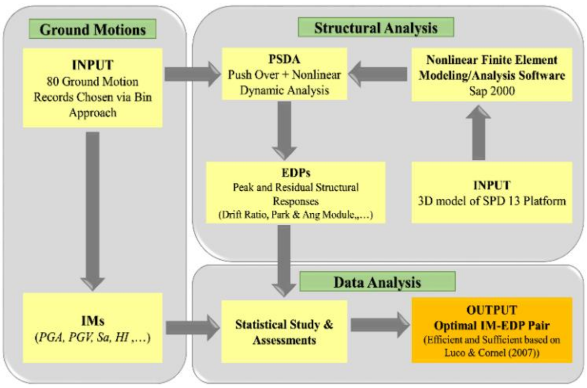


Fig. 1: Overview of the PSDM Evaluation Process

The drawn findings demonstrate that the quantitative comparison of results (the *RSM*) based on Kullback-Leibler divergence, for the studied IMs agrees well with previous qualitative-based conclusions based on Luco and Cornell's sufficiency criterion. Besides, the optimal PSDMs for further application in seismic assessment and fragility analysis of typical South Pars fixed pile-founded offshore platforms located in the Persian Gulf are proposed.

2. Probabilistic seismic demand model

Proposing a relationship between engineering demand parameters (EDPs) placed upon a structure or its members, and ground motion intensity measures (IMs), PSDM is provided based on probabilistic seismic demand analysis (PSDA). A PSDA is utilized to estimate the mean annual frequency (ν) of exceeding a given structural engineering demand parameter *(EDP > edp)* in a selected hazard environment *(IM > im)*, expressed as follows [6]:

$$\nu(EDP \geq edp) = \int_{edp} G(EDP \geq edp|IM = im)|d\lambda(im)|, \tag{1}$$

Where $G(EDP \geq edp|IM = im$ is the demand model that predicts the exceeding probability of an engineering demand parameter (*edp*) for a seismic hazard intensity measure (*im*). $\lambda(im)$ is the seismic hazard model to predict the annual exceeding probability of seismic hazard intensity measure (*im*) in a seismic hazard environment.
The basic formulation for probabilistic assessment of structural demands in which the conditional seismic demand is modeled employing a lognormal distribution, has been addressed by Cornell et al. [23]:

$$P[EDP \geq edp|IM] = 1 - \Phi\left(\frac{\ln(edp) - \ln(\eta_{EDP|IM})}{\beta_{EDP|IM}}\right) \tag{2}$$

In Eq. (2), $\Phi(\cdot)$ is the standard normal cumulative distribution function, *edp* is the peak or residual demand, $\eta_{EDP|IM}$ is the median value of the demand in terms of an IM, and $\beta_{EDP|IM}$ is the logarithmic standard deviation, or dispersion, of the demand conditioned on the IM. The relationship between median structural demand, $\eta_{EDP|IM}$, and IM can be estimated by a power model expressed in Eq.(3):

$$\eta_{EDP|IM} = a\, IM^b \tag{3}$$

Where constants *a* and *b* are regression parameters. Displayed in Eq. (4), Eq. (3) can be further transformed into the lognormal space:

$$\ln(\eta_{EDP|IM}) = \ln(a) + b \times \ln(IM) \tag{4}$$

In Eq. (4) the constant *b* is the slope and ln(*a*) is considered as the vertical intercept. Data for the regression analysis are developed by performing non-linear time history analyses with analytical fixed pile-founded offshore platform models representative of a typical offshore platform class using a suite of *N* ground motions. Peak demands (edp_i) are then plotted against the ground motion intensity for estimating the regression parameters, as well as the dispersion term ($\beta_{EDP|IM}$). According to Eq. (4), the conditional standard deviation of the regression used to estimate the dispersion, where edp_i is the *i*th realization of the demands from the non-linear time history analyses, can be shown as:

$$\beta_{EDP|IM} \approx \sqrt{\frac{\sum_1^N(\ln(edp_i) - (\ln(a.IM^b))^2}{N - (m+1)}} \tag{5}$$

In the case of having scalar IMs, a simple logarithmic linear regression model can be applied (i.e., m = 1). It should be recalled that:

$$\beta_{EDP|IM} = \sigma_{\ln EDP|IM} \tag{6}$$

Where $\sigma_{\ln EDP|IM}$ is the corresponding standard deviation in the arithmetic scale. The PSDM in the log-normally transformed space besides the parameters estimated from the regression analysis are illustrated in Figure 2.

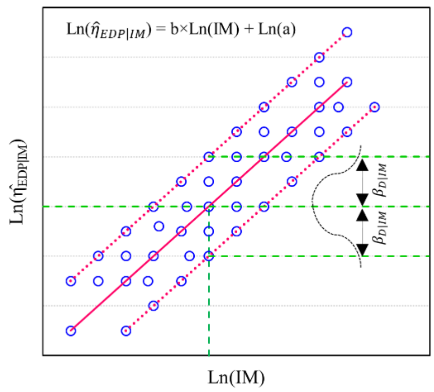


Fig. 2: PSDM illustration in lognormal space

From the above-presented formulation for the PSDM, it is evident that the selection of an appropriate IM can considerably lead to the ability of the model to capture the relationship. Moreover, the undue introduction of additional uncertainties can be clearly reduced.

2.1 Sufficiency of an Intensity Measure in Absolute Sense

Quantified by the *p*-value [23-25], a sufficient IM, can be defined as the one which is independent of ground motion characteristics, such as magnitude (M_w) and source distance (*R*) [5,6]. In statistics, the p-value is the probability of obtaining results at least as extreme as the observed results of a statistical hypothesis test, assuming that the null hypothesis is correct. The p-value is used as an alternative to rejection points to provide the smallest level of significance at which the null hypothesis would be rejected. The rejection of the null hypothesis does not tell us which of any possible alternatives might be better supported. In this context and for the objectives of this study, the null hypothesis refers to the slope coefficient of linear regression to be zero. Among widely used levels of significance which are 0.1, 1, and 5%, this study adopts the significance level of 5% (p-value = 0.05) for the threshold (the most common threshold in engineering applications). For p-values< 0.05, evidence for rejecting the null hypothesis is strengthening, and consequently, the IM which leads to p-values< 0.05, is considered as an insufficient IM. Based on the p-value's statistical definition and its usage for IM sufficiency evaluation [5], this sufficiency measure is regarded as an absolute measure owing to the information provided by this measure which is only in an absolute sense. Therefore, the sufficiency measure evaluated by the p-value only expressed whether an IM is sufficient or not. In General, an IM is sufficient in its absolute sense if and only if the probability distribution for demand parameter EDP given IM is independent of the ground motion acceleration time histories denoted as $\ddot{u}_g$:

$$p_{EDP|IM,\ddot{u}_g}(edp|IM(\ddot{u}_g)\ddot{u}_g = p_{EDP|IM}(edp|IM(\ddot{u}_g)) \quad (7)$$

Given IM, there is no additional information that can be provided by the ground motion time history, $\ddot{u}_g$, necessary for prediction of EDP. This is clearly a very strong condition, and it is unlikely that any scalar or low-dimensional vector IM satisfies it. Furthermore, even if the equality holds for a certain IM, it is not at all straightforward to demonstrate that it holds. Consequently, a simpler method has been proposed to represent the relative sufficiency of one IM with respect to another.

2.2. Sufficiency of an Intensity Measure in Relative Sense

With regard to recommending a measure of relative sufficiency of one IM with respect to another, Jalayer et al. [14] have utilized the concept of the Kullback–Leibler divergence, also known as the relative entropy [13]. The relative entropy presents a quantified measure for the ''distance'' between two probability distributions. In other words, the farther away it is from zero, the less sufficient (less informative) the IM is about the considered EDP [14]. The Kullback–Leibler divergence $\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM}\right)$ between the two probability density functions (PDF) $p_{EDP}|\ddot{u}_g$ and $p_{EDP}|IM$ can be achieved through Eq. (8). The difference between relative entropies $\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM}\right)$ in Eq. (8) is a functional of $\ddot{u}_g$. Its expected value over all the ground motions that could happen at the site is defined as the relative sufficiency measure for EDP of IM_2 relative to IM_1. In other words, $\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM}\right)$ measures on average how much information is lost on demand EDP by adopting the intensity measure, IM, instead of the entire ground motion time history $\ddot{u}_g$. Based on this definition, the difference in relative entropies $\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM_1}\right)$ and $\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM_2}\right)$ can be obtained as indicated in Eq. 9:

$$\mathcal{D}\left(p_{EDP\ddot{u}_g}\|p_{EDP|IM}\right) = \int_{\Omega_{EDP}} p_{EDP|\ddot{u}_g}(edp|\ddot{u}_g)\log_2\frac{p_{EDP|\ddot{u}_g}(edp|\ddot{u}_g)}{p_{EDP|IM}(edp|IM(\ddot{u}_g))}dy \quad (8)$$

$$\mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM_1}\right) - \mathcal{D}\left(p_{EDP|\ddot{u}_g}\|p_{EDP|IM_2}\right) = \int_{\Omega_{EDP}} p_{EDP|\ddot{u}_g}(edp|\ddot{u}_g)\log_2\frac{p_{EDP|IM_2}(edp|IM_1(\ddot{u}_g))}{p_{EDP|IM_1}(edp|IM_2(\ddot{u}_g))}dedp \quad (9)$$

$$I(EDP|IM_2|IM_1) = \int_{\Omega_{\mathrm{ag}}}\left[\int_{\Omega_{EDP}} p_{EDP|\ddot{u}_g}(edp|\ddot{u}_g)\log_2\frac{p_{EDP|IM_2}(edp|IM_1(\ddot{u}_g))}{p_{EDP|IM_1}(edp|IM_2(\ddot{u}_g))}dedp\right]\mathrm{p}_{\ddot{u}_g}(\ddot{u}_g)d\ddot{u}_g \quad (10)$$

$$I(EDP|IM_2|IM_1) = \int_{\Omega_{\ddot{u}_g}}\log_2\frac{p_{EDP|IM_2}(edp|IM_1(\ddot{u}_g))}{p_{EDP|IM_1}(edp|IM_2(\ddot{u}_g))}\mathrm{p}_{\ddot{u}_g}(\ddot{u}_g)d\ddot{u}_g \quad (11)$$

Jalayer et al. have defined the relative sufficiency of two alternative IMs, IM_1 and IM_2 as the expected value of their Kullback–Leibler divergence defined in Eq. (9) over all possible ground motion time histories. For a given $\ddot{u}_g$, edp_i is known and is equal to $edp_i(\ddot{u}_g)$; hence, the probability distribution Function (PDF), $p_{EDP|\ddot{u}_g}$, reduces to the Dirac delta function $\delta[edp_i(\ddot{u}_g)]$). Therefore, the equation can further be simplified. They also proposed a refined method for calculating the integral in Eq. (11) through Monte Carlo Simulation by adopting a stochastic ground motion model in conjunction with de-aggregation of the seismic hazard at the site. In addition, they argued that the relative sufficiency measure (*RSM*) can be approximately calculated by replacing the expectation with an average over a suite of *n* real ground motion records. In this regard, $EDP = \{edp_i, i = 1:N\}$ are the demand values for a suite of *N* ground motions (obtained from PSDA analysis). This provides a preliminary ranking of candidate IM_2 with respect to the reference IM_1.

Besides, the probability distribution function (PDF) $p_{EDP|IM}(edp_i|\text{IM})$ can be calculated by considering a lognormal distribution with the parameters defined in Eqs. (4) and (6):

$$p_{EDP|IM} = \frac{1}{edp\beta_{EDP|IM}}\phi\left(\frac{\ln(edp_i) - \ln(\eta_{EDP|IM})}{\beta_{EDP|IM}}\right) \qquad (12)$$

Where $\phi(\cdot)$ is the standardized Gaussian PDF. Hence, the relative sufficiency measure (*RSM*) can approximately be expressed as:

$$I(edp|IM_2|IM_1) \approx \frac{1}{N}\sum_{i=1}^{N}\log_2\left(\frac{\beta_{EDP|IM_1}}{\beta_{EDP|IM_2}}\frac{\phi\left(\frac{\ln(edp_i) - \ln(\eta_{EDP|IM_2})}{\beta_{EDP|IM_2}}\right)}{\phi\left(\frac{\ln(edp_i) - \ln(\eta_{EDP|IM_1})}{\beta_{EDP|IM_1}}\right)}\right) \qquad (13)$$

The relative sufficiency measure (*RSM*) of IM_2 with respect to IM_1 quantifies on average how much more information IM_2 relays to the designated structural response parameter about the ground motion with respect to IM_1. According to this approach, if $I(edp|IM_2|IM_1)$ is positive, it means that on average IM_2 is more sufficient, since it provides more information about the demand value. While the proposed method provides a relative ranking for IM sufficiency, the absolute assessment of sufficiency is based upon the *p*-value that has been addressed above.

3. Model Description

In this study, the class of four-leg fixed pile-founded offshore platforms have been selected to obtain the optimal probabilistic seismic demand modeling of these types of structures. This class of offshore platforms is one of the commonly installed/designed to be installed platforms found in the South Pars Oil and Gas Field located in the Persian Gulf. Aforementioned platforms generally consist of the following main parts: 1. A superstructure providing deck space for supporting operational appurtenances and other loads. 2. Welded tubular space frame, which is completely braced, extending from an elevation at or near the sea bed to above the water surface, and ~~is~~ designed to serve as the main structural element of the platform, transmitting lateral and vertical forces to the foundation (jacket). 3. Foundation elements such as piles, that permanently anchor the platform to the ocean floor, and carry both lateral and vertical loads. The key characteristics of the studied model (SPD 13 jacket) are presented in Figure 3.

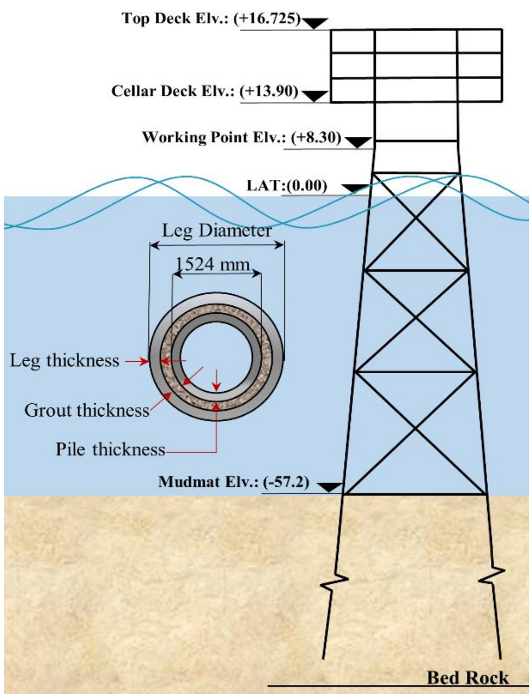


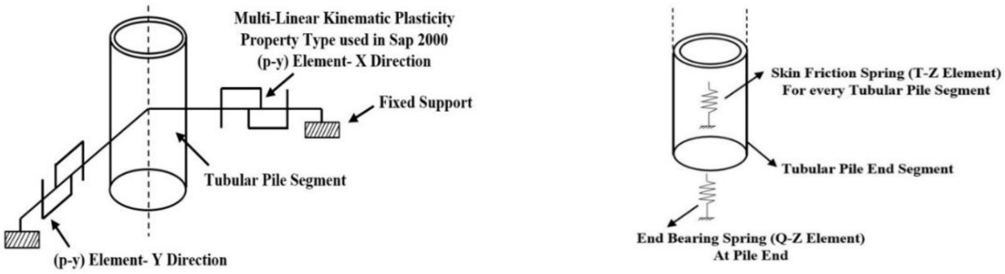
Fig. 3: A schematic 2D view of SPD 13 platform

3.1 The Pile Surrounded Soil Layer Characterization

According to the field and laboratory investigation, the stratum encountered at the borehole performed at the platform location was very soft calcareous becoming carbonate clay (CH) overlying medium dense and becoming loose clayey siliceous carbonate sand (SC) at 10.60m. More detailed of the pile surrounded soil profile is presented in APPENDIX A, Table A-1.

3.2 Modeling of piles

Structural behavior of offshore platform in the nonlinear range depends primarily on the soil–pile–structure interaction (SPSI). Several simplified methods have been attempted to capture the main aspects of SPSI, considering the fact that the seismic performance of soil and pile foundations is a complex issue. Utilized in the present study, the Beam on Nonlinear Winkler Foundation (BNWF) model, has been of a major concern [27]. Often referred to as the p-y method, this model uses parallel nonlinear soil–pile springs along the pile penetration length to approximate the interaction between the pile and the surrounding soil [28,29]. The p–y approach is utilized to model the lateral stiffness of soil. 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. In this study, p–y curves-generated based on the recommendation of API [26], utilized actual soil data according to the platform site geotechnical report. Sap 2000 [27], multi-linear plastic type link element, is employed in the numerical model proposed in this paper in order to model the nonlinear lateral relation between the soil and the pile. The p–y curve – in the proposed link element- defines the nonlinear link stiffness for the axial degree of freedom. The selected property models the hysteresis of the non-gapping soil behavior. Besides lateral loads, the pile foundation is exposed to the static and cyclic axial loads. Nonlinear axial load-deformation behavior along the shaft of driven tubular pile may be modelled using t-z data, recommended by API RP 2A-WSD [28]. Furthermore, the nonlinear load-displacement relationships and spring parameters (q-z data) for the studied platform location are also generated based on the recommendations contained in API RP 2A-WSD [28] according to the site investigation and pile testing data. The schematic configuration of the proposed model in SAP2000 is illustrated in Figure 4.



(a). P-Y Elements (b). T-Z and Q-Z Elements

Fig. 4: Schematic configuration of pile segments and (a) P-Y elements, (b) T-Z & Q-Z elements

To model the behavior of a pile, frame elements are chosen from the library of the SAP2000. The outer diameter of the pile is uniformly 1524 mm and penetrates into 110 m in the soil layers as listed in Table A-1. Dividing piles along their vertical axis, make the structure–pile–soil interaction simulation through several layers of different soils, feasible.

3.3 Seismic site response analysis

Achieved from a fundamental model, the soil response describes the soil cyclic behavior. It should be noted that, even at relatively small strains, soil exhibit small nonlinear behavior. Consequently, incorporating soil nonlinearity in any site response analysis, is essential. Each soil layer is characterized by its thickness, mass density, shear wave velocity, and nonlinear soil properties including nonlinear modulus reduction and damping curves which affect the selected ground motion records. In fact, the results of site response display seismic performance assessment of nonlinear ground response analysis within the soil profile. They can reflect the input ground motions determination of uncertainties, the site velocity profile characterization and the nonlinear properties specification as well as analysis technique selection [31]. The computer program DEEPSOIL is employed to perform site response simulations based on the soil layer characteristics and selected ground motion records [32]. This program performs nonlinear site response analysis using outcropping motions in the time domain and the layered soil column as a multiple-degree-of-freedom lumped mass system.

Figure 5 shows the multi-degree-of-freedom lumped parameter model for layered soil in which displacement time history from nonlinear site response analysis is also considered.

The dynamic model of fixed pile-founded offshore platforms should reflect the key analytical parameters of mass, damping, and stiffness. The mass consideration would be accurate if all deck loads, conductors, and appurtenances, as well as the mass of platform steel and water enclosed in submerged tubular members (added mass) and the mass of marine growth expected to accumulate on the structure (increased member diameter due to marine growth) have been included [30]. The three dimensional (3D) model was created employing Sap 2000, with drag and inertia forces exerted and Morison Equation used to calculate hydrodynamic loads. Each analysis routine includes a modal analysis to determine natural frequency as well as mode shape information, a static pushover analysis (SPO) to represent yield values, and a nonlinear dynamic time-history analysis (NTH) to determine EDPs.

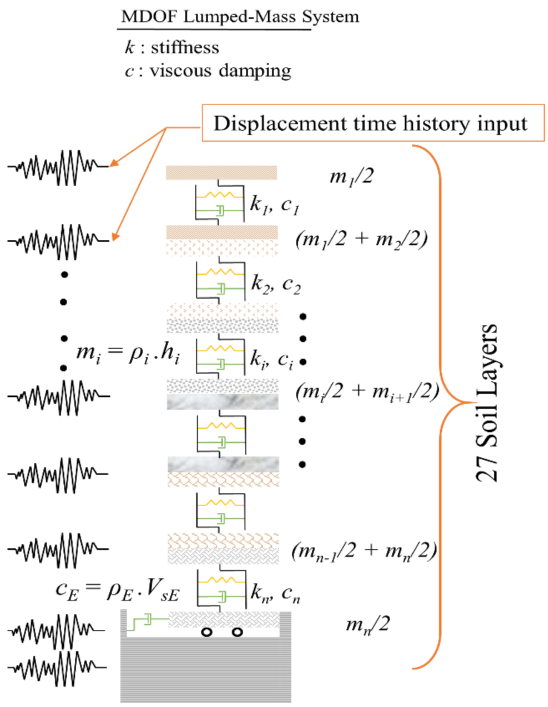


Fig. 5: Seismic Site Response Consideration

3.4 Frames and Mass Consideration

The non-structural members such as flooding system, centralizer, pad-eyes, plates and stiffeners, etc. are classified as the platform appurtenances. The analysis models include only the major structural components, and the contribution of the conductors to the platforms' stiffness and strength is neglected. The frame elements, considered as jacket horizontal members, are rigidly connected at the ends. The numerous quantities were monitored in order to extract maximum and residual dynamic quantities, such as axial force, moment, horizontal and vertical displacements and so on. Access to response quantities extracted from the model is provided by post-processing. The periods of the first three vibration modes for the studied platform is listed in Table 1. Evident from the table, Sap 2000 results match well with the characteristics obtained from the initially designed model of SPD 13 employing Structural Analysis Computer System (SACS) software.

Table. 1: Comparison of the First Three Periods in Sap 2000 and SACS

	Period	Sap 2000(s)	SACS(s)
SPD 13, 3D Model, Modal Analysis	1st Mode	2.26	2.38
	2nd Mode	1.96	2.03
	3rd Mode	1.51	1.43

4. Ground Motion Records and Intensity Measures for PSDA

4.1 Record Selection

Not being based on seismic hazard curves, PSDA uses a ground motion bin approach instead [3,33]. It would also be possible to perform the analysis using a standard Monte Carlo simulation [34] involving thousands of ground motions, or by generation of synthetic ground motions. The bin approach chooses a suite of ground motions typical for the region under study from a database of recorded ground motions. In this study, four bins with 20 non-near-field ground motions, each obtained from the PEER Strong Motion Database have been selected [35].

The delineation between small (SM) and large (LM) magnitude bins was at Mw = 7. Ground motions with closest distance R ranging between 20 and 50 km were grouped into a small distance (SR) bin, while ground motions with R > 50 km up to 80 km were in the large distance (LR) bin. All ground motions were recorded on NEHRP soil type D sites [36]. The details of all ground motion records, the name of earthquakes, sensor location, magnitude and distance are presented in APPENDIX A, Table A-2.

4.2 Intensity Measure Selection

The set of IM candidates under investigation in this study is identical to that used earlier by Wang et al. [12] (26 IM candidates). Among the IM candidates, 14 have been presented here based on the results deduced through preliminary comprehensive assessments indicating that most of the acceleration-related IMs are generally inefficient. The fact that acceleration-related IMs do not result in optimal PSDMs for structures with lower natural frequencies (i.e. with periods of more than 0.5 s) has been also mentioned in some of previous studies. Besides, high-rise buildings studies indicated that due to high-rise response frequency range, which is much wider than low-rise or mid-rise buildings, IMs such as spectral values $S_a(T_1)$, $S_v(T_1)$, $S_d(T_1)$ and $PS_v(T_1)$ represent only specific points in frequency content of the response spectrum [4-6], [37-39]. For that reason, intensity measures comprising a wider range of frequency content of response spectra (e.g. *HI*) are more appropriate for the case of structures with periods of more than 0.5 seconds (such as high-rise buildings and fixed pile-founded offshore platforms). This set of IMs was primarily categorized and is outlined in Table 2.

5. PSDM Evaluations and Comparison Results of IMs

Efficiency Evaluations: Dispersion of IM–EDP pairs is estimated by Equation 5 and by calculating the edp_i based on the linear or piecewise-linear regression fit in a log–log space. An optimal IM would be distinguished by smaller values of dispersion ($\beta_{EDP|IM} = \sigma_{\ln EDP|IM}$) for IM-EDPs. Figure 6 indicates the IM efficiency for global, intermediate and local EDPs.

The studied EDPs are global drift ratio (θ_{Global}) and global ductility (μ_{Global}), which describe the platform global response. Besides, for intermediate level EDPs, jacket part drift ratio (θ_{jacket}) and mudline elevation differential settlement ($Z_{mudline}$)are considered. Likewise, the working point elevation drift ratio ($\theta_{w.p.}$) and top deck elevation differential settlement ($Z_{top\ deck}$) have been issued for local term considerations. It is worth noting that, EDP selection issued by Asgarian et al. besides El-Din and Kim, has been also considered [40,41].

As mentioned, the estimated dispersion $\sigma_{\ln EDP|IM}$ serves as quantitative measure for predictive efficiency of the candidate IM. While IMs resulting in standard errors in order of 0.20-0.30 are superb, the range 0.30-0.40 is still considered as reasonably acceptable [42]. Accordingly, and as illustrated in Figure 6, all PSDMs are considered as reasonably efficient; however, those that resulted in lower values of $\sigma_{\ln EDP|IM}$, rose to the higher ranks and were considered as more efficient.

Sufficiency Evaluations: A linear regression analysis of the residuals of *lnEDP|IM* relative to the ground motion parameters has been carried out for evaluating the IM sufficiency. Based on this assessment, the significance of having a linear trend, measured by *p*-value, implies the sufficiency/insufficiency associated with the desired IM. The *p*-value of 5% is considered as the cutoff for an insufficient IM since this value is an appropriate threshold for engineering investigations (i.e., IMs with *p*-values of less than 0.05 are assumed as insufficient IMs). Figure 7 shows samples for the linear regression of the residues versus M_w and R for different EDPs. Besides, the *p*-values of all the fourteen IMs with respect to M_w and R, in terms of global, intermediate and local EDPs have been calculated and illustrated in Figure 8 for the model of fixed pile-founded offshore platform. The *p*-value cutoff (0.05) is identified by the horizontal line in each plot.

Table. 2 : IM Candidates

	Intensity Measure	
	Notation	Name
Non-Structure Specific	**Acceleration-Related**	
	I_a	Arias Intensity
	I_c	Characteristic Intensity
	CAV	Cumulative abs. velocity
	Velocity-Related	
	PGV	Peak ground vel.
	V_{rms}	RMS of vel.
	SMV	Sustained maximum vel.
	SED	Specific energy density
	Displacement-Related	
	D_{rms}	RMS of disp.
	Time-Related	
	V_{max}/A_{max}	Peak vel./acc. ratio
Structure Specific	**Acceleration-Related**	
	$S_a(T_1,5\%)$	Spectral acc.
	Velocity-Related	
	$S_v(T_1,5\%)$	Spectral vel.
	HI	Housner intensity
	VSI	Vel. spectrum intensity
	Displacement-Related	
	$S_d(T_1,5\%)$	Spectral disp.

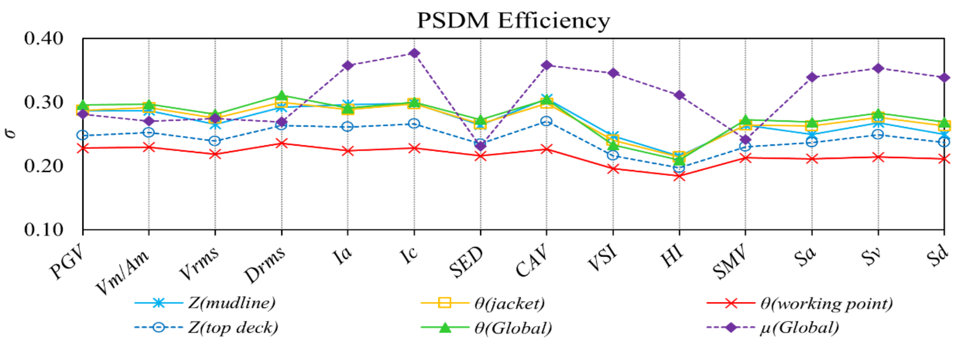


Fig. 6: PSDM Efficiency

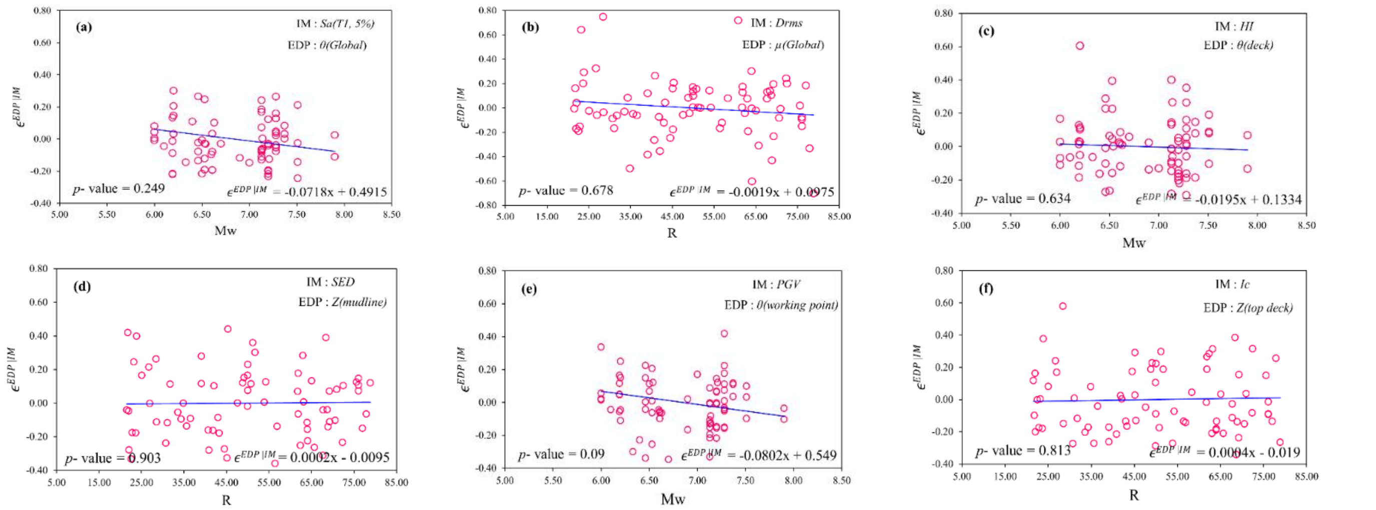
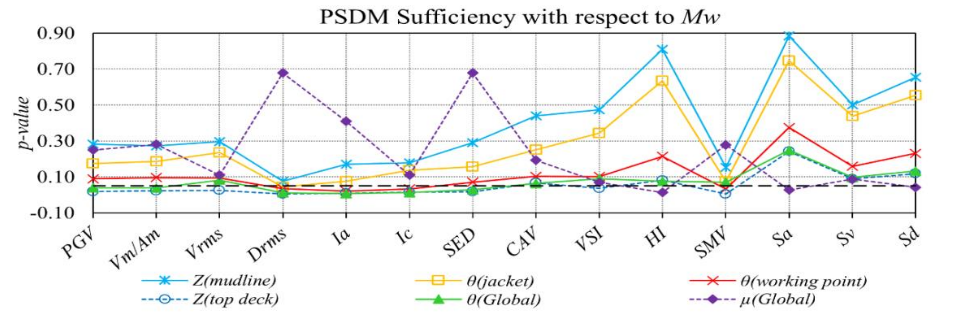
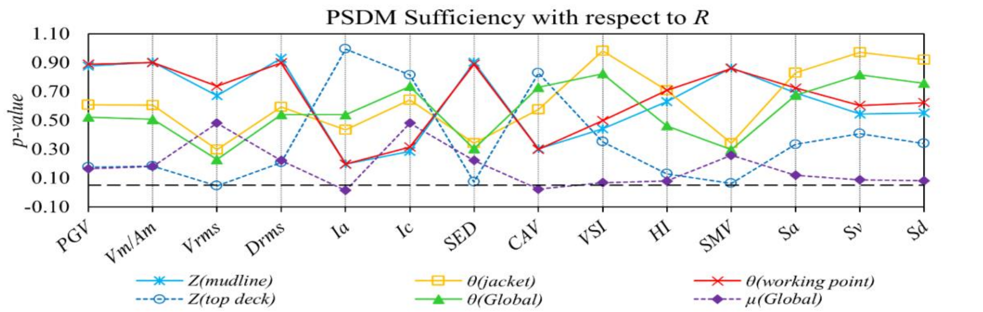


Fig. 7: Linear regressions of the residues, ϵ EDP|IM, for a PSDMs versus magnitude(M_w) and source distance(R)



8a. Sufficiency Assessment with respect to M_w



8b. Sufficiency Assessment with respect to *R*

Fig. 8: PSDM Sufficiency Assessment with respect to a. M_w & b. *R*

Furthermore, Table 3 shows the sufficiency comparison of the PSDMs conditioned by fourteen IM candidates and six EDPs. Referring to Table 3 and as illustrated in Figure 8, for PSDMs provided by two global EDPs, θ_{Global} and μ_{Global}, the following conclusions can be drawn:

- The *p*-values with respect to *R* and M_w varies clearly for PSDMs provided by each EDPs.
- The sufficient IM candidates in case of μ_{Global} conditioned PSDMs reveal that, among fourteen IMs, I_a and *CAV* are not sufficient with respect to *R*, owing to *p*-values being less than 0.05, however *HI*, $S_a(T_1, 5\%)$ and $S_d(T_1, 5\%)$ are insufficient IMs with respect to M_w. Consequently, I_a, *CAV*, *HI*, $S_a(T_1, 5\%)$ and $S_d(T_1, 5\%)$ do not lead to optimal PSDMs provided by μ_{Global}.

Besides, based on the results in terms of intermediate EDPs (θ_{jacket} and $Z_{mudline}$), it can be concluded that:

- The *p*-value based sufficiency measure is not consistent with respect to *R* and M_w. As an example, for PSDMs provided by θ_{jacket}, the IMs show strong evidence to be sufficient with respect to *R*, are *VSI* and $S_v(T_1, 5\%)$.
- All 14 IMs result in sufficient PSDMs for both θ_{jacket} and $Z_{mudline}$ with respect to *R*.
- PSDMs conditioned by $Z_{mudline}$ for all fourteen IMs are considered sufficient with respect to M_w, owing to *p*-values being larger than 0.05.

- The similar trend is followed by PSDMs provided by both θ_{jacket} and $Z_{mudline}$ for sufficiency assessment with respect to M_w.

Finally, the sufficiency assessment of PSDMs conditioned by $\theta_{working\ point}$ and $Z_{top\ deck}$ with respect to R and M_w, indicated that:

- All fourteen IMs, have led to sufficiency with respect to R, due to p-value being higher than 0.05.
- Among all IM candidates, for working point drift ratio, D_{rms}, I_a, I_c and *SMV* appear to be insufficient with respect to M_w.
- However, only $S_a(T_1, 5\%)$, *CAV*, *HI*, $S_v(T_1, 5\%)$ and $S_d(T_1, 5\%)$ yield to sufficient PSDMs conditioned by $Z_{top\ deck}$, with respect to M_w.

Table. 3a: Sufficiency comparison using p-values (absolute sense) with respect to M_W

		Global		Intermediate		Local	
		θ_{Global}	μ_{Global}	$Z_{mudline}$	θ_{jacket}	$\theta_{w.p.}$	$Z_{top\ deck}$
PSDM Sufficiency w. r. to M_w	*PGV*	0.039	0.249	0.282	0.174	0.090	0.019
	V/A	0.038	0.281	0.272	0.187	0.096	0.023
	V_{rms}	0.083	0.110	0.296	0.236	0.093	0.026
	D_{rms}	0.011	0.678	0.077	0.044	0.034	0.005
	I_a	0.007	0.409	0.171	0.076	0.021	0.010
	I_c	0.013	0.110	0.178	0.137	0.034	0.015
	SED	0.029	0.678	0.291	0.156	0.070	0.018
	CAV	0.064	0.193	0.441	0.252	0.103	0.067
	VSI	0.090	0.070	0.474	0.343	0.103	0.036
	HI	0.076	0.013	0.810	0.634	0.215	0.082
	SMV	0.071	0.278	0.152	0.075	0.033	0.005
	$S_a(T_1,5\%)$	0.249	0.028	0.884	0.747	0.374	0.241
	$S_v(T_1,5\%)$	0.098	0.087	0.500	0.437	0.159	0.089
	$S_d(T_1,5\%)$	0.133	0.042	0.653	0.555	0.231	0.118

Table. 3b: Sufficiency comparison using p-values (absolute sense) with respect to R

		Global		Intermediate		Local	
		θ_{Global}	μ_{Global}	$Z_{mudline}$	θ_{jacket}	$\theta_{w.p.}$	$Z_{top\ deck}$
PSDM Sufficiency w. r. to R	***PGV***	0.522	0.164	0.875	0.606	0.889	0.175
	V/A	0.506	0.179	0.903	0.605	0.899	0.182
	V_{rms}	0.227	0.480	0.670	0.297	0.736	0.046
	D_{rms}	0.539	0.221	0.928	0.592	0.897	0.206
	I_a	0.537	0.014	0.197	0.434	0.195	0.993
	I_c	0.734	0.480	0.284	0.642	0.314	0.813
	SED	0.302	0.221	0.903	0.341	0.888	0.072
	CAV	0.728	0.023	0.301	0.575	0.298	0.826
	VSI	0.822	0.067	0.440	0.980	0.498	0.351
	HI	0.461	0.078	0.629	0.707	0.708	0.129
	SMV	0.296	0.257	0.864	0.340	0.859	0.063
	$S_a(T_1,5\%)$	0.673	0.119	0.690	0.829	0.723	0.332
	$S_v(T_1,5\%)$	0.814	0.087	0.542	0.971	0.602	0.407
	$S_d(T_1,5\%)$	0.756	0.081	0.549	0.920	0.622	0.339

In general, since the sufficiency assessment of PSDMs based on p-value measures is not consistent, it is difficult to rank the sufficiency of IMs based on the hypothesis testing for dependence of the residues between actual responses and estimated demands upon R and M_w. In other words, as the p-value based sufficiency measure only provides the information in absolute sense; exhibiting the preference of one IM compared to others is not possible. Jalayer et al. [14] stated that, evaluating sufficiency in a relative sense for a scalar or low-dimensional vector IM, is more reasonable; i.e., to investigate whether one IM is more sufficient (more informative) compared to another IM for predicting a structural response parameter. As mentioned, the relative sufficiency measure (*RSM*), is based on the Kullback-Leibar divergence which evaluates the sufficiency of one IM with respect to another, without further information on magnitude and source distances. For the objectives of this study, since $S_a(T_1, 5\%)$ has been the commonly used IM for seismic fragility assessments of fixed pile-founded offshore platforms, it has been employed as the reference IM (IM_1) and the relative sufficiency measure, $I(edp|IM_2 \mid S_a(T_1, 5\%))$, has been calculated. The results reveal how many extra bits of information, on average, the candidate IM gives about the desired EDP compared with $S_a(T_1, 5\%)$. A positive value of *RSM* indicates that on average, IM_2 provides more information (i.e., is more sufficient) than $S_a(T_1, 5\%)$, while a negative value means that IM_2 provides on average less information (i.e., is less sufficient) compared to $S_a(T_1, 5\%)$ for predicting the demand parameter of interest. Owing to the fact that *RSM* provides the preliminary ranking of IM sufficiency among other IM candidates, it is worth noticing that relative sufficiency does not show the absolute result whether an IM is sufficient or not. Thus, both sufficiency and relative sufficiency assessment for PSDMs provided by IM candidates are necessary. The results of relative sufficiency of thirteen IM candidates with respect to $S_a(T_1, 5\%)$ have been given. For PSDMs in terms of global, intermediate and local *RSM* have been calculated and plotted in Figure 9. Besides, as a quantitative tool, relative sufficiency measures are listed in Table 5 which facilitate better understanding of the concept.

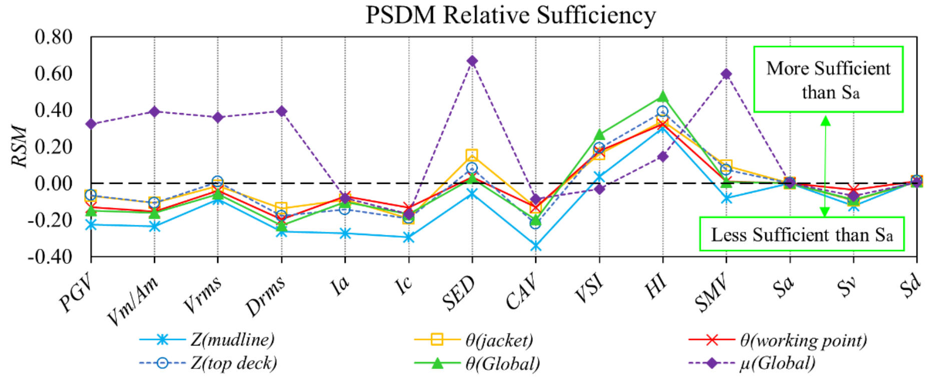


Fig. 9: Relative Sufficiency Assessments of IMs with respect to $S_a(T_1, 5\%)$

Table. 4: Relative Sufficiency Measure based on Kullback-Leibler Divergence

		Global		Intermediate		Local	
		θ_{Global}	μ_{Global}	$Z_{mudline}$	θ_{jacket}	$\theta_{w.p.}$	$Z_{top\ deck}$
Relative Sufficiency Measure	***PGV***		0.323	-0.226	-0.069	-0.132	
	V/A		0.391	-0.235	-0.107	-0.155	
	V_{rms}	-0.059	0.361	-0.087	-0.014	-0.041	
	D_{rms}		0.394	-0.265			
	I_a			-0.272	-0.090		
	I_c		-0.169	-0.294	-0.193		
	SED		0.667	-0.058	0.151	0.034	
	CAV	-0.199		-0.340	-0.133	-0.132	-0.222
	VSI	0.266	-0.032	0.036	0.160	0.175	
	HI	0.476		0.302	0.338	0.322	0.390
	SMV	0.007	0.597	-0.080	0.095		
	$S_a(T_1,5\%)$	**0.000**		**0.000**	**0.000**	**0.000**	**0.000**
	$S_v(T_1,5\%)$	-0.089	-0.070	-0.123	-0.095	-0.035	-0.092
	$S_d(T_1,5\%)$	0.007		0.013	0.008	0.012	0.014

Based on the results listed in Table 5 (not including insufficient pairs) and illustrated in Figure 9, it can be concluded that:

- For PSDMs provided by θ_{Global}, as a global EDP, among the IM candidates which stand above the horizontal dashed line (showing positive *RSM*), *VSI* and *HI* are the most sufficient IMs, while *SED*, *SMV* and $S_d(T_1, 5\%)$ have just passed the dashed line.
- In μ_{Global} conditioned PSDMs, I_a, I_c, *CAV*, *VSI* and $S_v(T_1, 5\%)$ are less sufficient IMs in comparison with $S_a(T_1, 5\%)$.
- Among more sufficient IMs compared with $S_a(T_1, 5\%)$, for μ_{Global} based PSDMs, *SED* and *SMV* show the highest relative sufficiency measure.
- A great harmony can be seen in *RSM* for both intermediate EDP provided PSDMs, θ_{jacket} and $Z_{mudline}$, the same trend is repeated for PSDMs based on local EDP, i.e., $\theta_{working\ point}$ and $Z_{top\ deck}$, too. However, this harmony is in IM ranking.

6. Results

Table 5 presents the PSDM ranking results on the relative sufficiency metric as well as considering the efficiency, to add up the established assessments and provide a comparative tool to simplify the selection of PSDMs which have good performance conditioned by EDPs in terms of global, intermediate and local.

It should be noticed that, for equal RSMs, efficiency is determinant. It can be drawn that, among 10 top PSDMs, most of the IMs belong to the category of velocity-related ones. Moreover, most of the optimal PSDMs are among those conditioned by global EDPs. However, one of the most remarkable and challenging results is definitely the distance between top PSDMs and $S_a(T_1,5\%)$-conditioned pairs; while this IM has been thoroughly used in seismic risk assessment and reliability-based studies of offshore platforms.

Table. 5: Optimal PSDMs Ranking

	Global	Intermediate	Local
1	$SED\text{-}\mu_{Global}$ *ß=0.23,RSM=0.67*		
2	$SMV\text{-}\mu_{Global}$ *ß=0.24,RSM=0.59*		
3	$HI\text{-}\theta_{Global}$ *ß=0.21,RSM=0.48*		
4			$HI\text{-}Z_{top\ deck}$ *ß=0.20,RSM=0.39*
5	$D_{rms}\text{-}\mu_{Global}$ *ß=0.27,RSM=0.39*		
6	$V/A\text{-}\mu_{Global}$ *ß=0.27,RSM=0.39*		
7	$V_{rms}\text{-}\mu_{Global}$ *ß=0.27,RSM=0.36*		
8		$HI\text{-}Z_{mudline}$ *ß=0.22,RSM=0.34*	
9			$HI\text{-}\theta_{w.p.}$ *ß=0.18, RSM=0.32*
10	$PGV\text{-}\mu_{Global}$ *ß=0.28,RSM=0.32*		

7. Conclusion

In the context of PBEE, optimal PSDM in general and sufficiency in particular for the typical fixed pile-founded offshore platforms of South Pars, has been assessed. For the purpose of this study, PSDA analysis is employed selecting a suite of 80 ground motion records to construct a lognormal probability distribution for describing the EDPs that is conditional on the adopted IM. The superiority of PSDMs has been evaluated based on efficiency and sufficiency considerations presented by Luco and Cornel [5]. Since there are major drawbacks involved with *p*-value based sufficiency, relative sufficiency of IM candidates [14] has been also assessed based on Kullback-Leibler Divergence. Derived from the information theory, Kullback-Leibler Divergence implemented here, has led to the comparison of the capability of IMs in predicting the structural response by providing extra bits of information the candidate IM gives about the desired EDP compared with the base IM. Based on the observations of this study, it can be drawn that all PSDMs perform well according to efficiency evaluations. Furthermore, among PSDMs provided by 14 IM and 6 EDP candidates (=84), most of optimal PSDMs are based on velocity-related IMs and global EDPs. Particularly, $SED\text{-}\mu_{Global}$ as well as $HI\text{-}\theta_{Global}$, are among top PSDMs. Accordingly, the findings are proposed to be considered for further studies on seismic risk assessments and fragility calculations of fixed pile-founded offshore platforms.

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APPENDIX A

Table A-1. Soil Layer Characteristics

Unit	Depth below Seafloor (m) From	To	Generic Soil Description	γ' (Kn/m³)
1a	0.00	3.50	Very Soft CLAY	5.8
1b	3.50	10.60	Very Soft CLAY	7.3
2a	10.60	12.60	Medium dense clayey siliceous carbonate SAND	9.4
2b	12.60	14.10	Loose clayey siliceous carbonate SAND	9.4
3	14.10	15.20	Firm CLAY	8.5
4	15.20	17.20	Medium dense silty siliceous SAND	8.9
5a	17.20	20.00	Firm to soft CLAY	7.5
5b	20.00	24.80	Soft to firm CLAY	8.8
6a	24.80	27.80	Stiff CLAY	9.5
6b	27.80	40.00	Stiff CLAY	8.8
6c	40.00	42.20	Stiff CLAY	9.6
7a	42.20	43.66	Very dense clayey siliceous carbonate SAND	10.0
7b	43.66	45.10	Dense clayey siliceous carbonated SAND	10.0
8	45.10	47.00	Hard sandy CLAY	9.5
9a	47.00	52.00	Very stiff CLAY	10.0
9b	52.00	53.50	Very stiff CLAY	10.2
10	53.50	54.00	Dense cemented siliceous carbonate SAND	10.2
11a	54.00	56.50	Very stiff CLAY	9.9
11b	56.50	57.50	Stiff CLAY	9.9
12a	57.50	59.70	Dense locally moderately cemented clayey siliceous carbonate GRAVEL	9.5
12b	59.70	61.60	Medium Dense Clayey siliceous carbonate GRAVEL	9.6
13a	61.60	79.50	Very stiff CLAY	9.5
13b	79.50	89.80	Very stiff CLAY	10.0
14	89.80	90.65	Very dense locally cemented clayey siliceous carbonate SAND	9.6
15a	90.65	98.00	Hard CLAY	9.5
15b	98.00	105.3	Hard CLAY	9.3
16	105.3	110.40	Hard CLAY	10.3

Table A-2. Ground Motion Records Characteristics

LMLR				LMSR			
Event	**M**	**R(km)**	**Station**	**Event**	**M**	**R(km)**	**Station**
Trinidad	7.20	76.06	Rio Dell Overpass-FF	Landers	7.28	34.86	Barstow
Trinidad	7.20	76.06	Rio Dell Overpass-E Ground	Landers	7.28	21.78	Desert Hot Spring
Trinidad	7.20	76.06	Rio Dell Overpass-W Ground	Landers	7.28	26.96	Mission Creek Fault
Landers	7.28	69.21	Amboy	Landers	7.28	23.62	Yermo Fire Station
Landers	7.28	62.98	Fort Irwin	Gulf of Aqaba	7.20	43.29	Eilat
Landers	7.28	68.66	Hemet Fire Station	Kocaeli, Turkey	7.51	31.74	Goynuk
Landers	7.28	54.25	Indio-Coachella Canal	Kocaeli, Turkey	7.51	30.73	Iznik
Kocaeli, Turkey	7.51	58.33	Hava Alani	Duzce,Turkey	7.14	34.30	Mudurnu
Kocaeli, Turkey	7.51	51.17	Mecidiyekoy	Duzce,Turkey	7.14	45.16	Sakarya
Caldiran,Turkey	7.21	50.78	Maku	Manjil, Iran	7.37	49.97	Qazvin
Manjil, Iran	7.37	75.58	Abhar	Hector Mine	7.13	41.81	Amboy
Manjil, Iran	7.37	63.96	Rudsar	Hector Mine	7.13	31.06	Joshua Tree
Hector Mine	7.13	64.08	Baker Fire Station	Hector Mine	7.13	42.06	Twenty nine Palms
Hector Mine	7.13	61.85	Big Bear Lake- Fire Station	Denali, Alaska	7.90	49.94	Carlo (temp)
Hector Mine	7.13	77.01	Cabazon	Denali, Alaska	7.90	42.99	R109 (temp)
Hector Mine	7.13	56.4	Desert Hot Spring	Landers	7.28	45.34	Forest Fall Post Office
Hector Mine	7.13	65.04	Fort Irwin	Landers	7.28	48.84	Indio - Jackson Road
Hector Mine	7.13	61.86	N.Palm Spring Fire Sta. #36	Landers	7.28	40.67	Morongo Valley Hall
El Mayor- Cucapah	7.20	72.44	Salton City	El Mayor- Cucapah-	7.20	28.53	El Centro – Meloland
El Mayor- Cucapah	7.20	67.71	Ocotillo Wells–Veh. Rec.	El Mayor- Cucapah-	7.20	22.83	El Centro Differential Array
SMLR				**SMSR**			
Event	**M**	**R(km)**	**Station**	**Event**	**M**	**R(km)**	**Station**
Borrego	6.50	56.88	El Centro Array #9	Northern Calif –01	6.40	44.52	Ferndale City Hall
Ierissos, Greece	6.70	65.67	Ierissos	Northern Calif –01	6.50	26.72	Ferndale City Hall
Morgan Hills	6.19	51.68	APPLE 1E-Hayward	Borrego Mtn	6.63	45.12	El Centro Array #9
Morgan Hills	6.19	63.16	Los Banos	Imperial Valley –06	6.53	23.17	Calipatria Fire Station
Morgan Hills	6.19	70.93	SF Intern. Airport	Imperial Valley –06	6.53	49.1	Coachella Canal #4
Big Bear –01	6.46	78.81	Featherly Park – Maint	Imperial Valley –06	6.53	22.03	Delta
Big Bear –01	6.46	67.74	Mt Baldy – Elementary Sch.	Imperial Valley –06	6.53	21.98	El Centro Array #13
Big Bear –01	6.46	64.04	Phelan – Wilson Ranch	Imperial Valley– 06	6.53	35.64	Niland Fire Station
Chi-Chi, Taiwan-04	6.20	50.02	CHY015	Victoria, Mexico	6.33	39.1	SAHOP Casa Flores
Chi-Chi, Taiwan04	6.20	60.77	TCU117	Morgan Hills-1984	6.19	39.08	Capitola
Chi-Chi, Taiwan-04	6.20	51.48	TCU118	Chalfant Valley–02	6.19	21.55	Benton
Chi-Chi, Taiwan-04	6.20	62.35	TTN044	Superstition Hills	6.54	23.85	Wildlife Liquifaction Arr.
Tottori, Japan	6.61	70.55	HRS010	Big Bear–01	6.46	47.6	Hemet Fire Station
Tottori, Japan	6.61	72.30	HRS015	Big Bear-01	6.46	40.87	North Palm Springs #36
Tottori, Japan	6.61	77.85	SMN017	Kobe, Japan	6.90	49.91	Chihaya
Bam, Iran	6.60	69.28	Jiroft	Kobe, Japan	6.90	22.5	Kakogawa
Parkfield-02, CA	6.00	61.72	San Luis Obispo	Chi-Chi, Taiwan 04	6.20	28.45	CHY034
Parkfield-02, CA	6.00	68.85	Cambria – Hwy 1Caltrans	Chi-Chi, Taiwan 04	6.20	36.48	TCU141
Parkfield-02, CA	6.00	53.87	KING CITY	Joshua Tree, CA	6.10	25.04	Indio – Jackson Road
Parkfield-02, CA	6.00	68.38	Greenfield – Police Station	Darfield, New Zealand	7.00	33.54	MAYC