**Performance-based Seismic Response of Pile Foundations**

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**ABSTRACT:** Investigation of the correlation between so-called engineering demand parameters (EDPs) and various intensity measures (IMs) has received substantial attention in structural earthquake engineering, as accurate prediction of seismic demand is desired in performance-based seismic evaluation of structures. Little work however has been performed quantifying the seismic response of pile foundations within a performance-based context. In this study the seismic demands of pile foundations are investigated in a performance-based approach. A simple soil-pile-structure model consisting of a two layer soil deposit with a single pile and a single-degree of freedom superstructure is used in a parametric study to determine the salient features in the seismic response of the soil-pile foundation system. A suite of 40 'ordinary' ground motion records were scaled to various ranges of intensity to probabilistically investigate the full range of pile behaviour, from initial elastic response to failure. Various IMs are used to inspect their correlation with the seismic demand, which is measured in terms of: peak lateral displacement of the pile head (relative to pile toe) and peak pile curvature. The lowest scatter in the prediction of pile response was observed when seismic intensity was measured in terms of velocity-based IMs.

## INTRODUCTION

Within the context of emerging trends in performance-based earthquake engineering (PBEE), seismic performance is measured with respect to the demand (and its associated consequences) of engineering systems during a seismic event, as opposed to the conventional factor of safety approach. The low – frequency high – consequence nature of seismic events coupled with randomness and uncertainty in response behaviour and modelling, requires that a probabilistic stance is also endeavored when seismic performance is assessed. In such a probabilistic framework system performance is principally affected by the uncertainty in each of the variables comprising the system, which are generally differentiated as being either aleatory or epistemic. Kramer and Mitchell (2006) give a good overview of the effect of uncertainties in PBEE.

Significant research over the past decade has focused on determining IMs which correlate well with the prediction of structural response due to ground motion excitation (e.g. Shome and Cornell, 1999; Baker and Cornell, 2005; Tothong and Luco, 2007). Recently, Kramer and Mitchell (2006) investigated the correlation of various IMs with the occurrence of liquefaction in a soil deposit. They found that compared to traditional ground motion parameters, such as PGA and Arias intensity, a new parameter, $CAV_5$, defined as the cumulative absolute velocity for velocities above 5 cm/sec provided a significantly reduced uncertainty in the prediction of peak pore pressure ratio (the EDP used in this case).

In this paper the performance-based response of pile foundations is investigated considering the correlation of various IMs with the seismic demand on pile foundations embedded in non-liquefiable soils. Firstly, discussion is given to the determination of which measures of demand (EDP) should be used to describe the seismic response of the pile(s). Next, various candidate IMs are examined and ranked based on their efficiency in predicting the EDP, and their independence from the rupture magnitude and propagation distance to the recorded site.

## PREDICTION OF SEISMIC DEMAND FOR PILES

It is well known that the seismic demands on pile foundations arise due to both inertial effects from the superstructure and kinematic effects imposed by cyclic lateral ground displacements (Gazetas and Mylonakis, 1998). The vibratory motion in the vicinity of the pile foundation is complex and differs significantly from the free-field motion due to the flexural rigidity of the piles, which causes refraction and scattering of the incident seismic waves. Prediction of seismic demands considering both inertial and kinematic effects requires a rigorous dynamic analysis of the soil-pile-structure system. Experience from recent strong earthquakes and observations from benchmarking experiments on piles have shown that pile foundations are subjected to very large lateral loads leading to serious damage and collapse of piles. Hence a key requirement in the analysis is estimation of the inelastic response and damage to the pile(s). While a pseudo-static analysis of a simple beam-spring model provides a convenient design-oriented approach for preliminary assessment of pile response, a dynamic time-step analysis based on the effective stress principle allows modelling of the salient features of soil-pile-structure interaction including the complex effects of excess pore pressures and soil non-linearity. This approach permits more accurate prediction of the seismic demand on piles and was therefore adopted in this study. Since assessment of pile response in the performance-based framework has not been scrutinized to date, first attention is given to the identification of adequate seismic demand and intensity measures for piles.

## OPTIMAL INTENSITY MEASURES FOR PILE RESPONSE

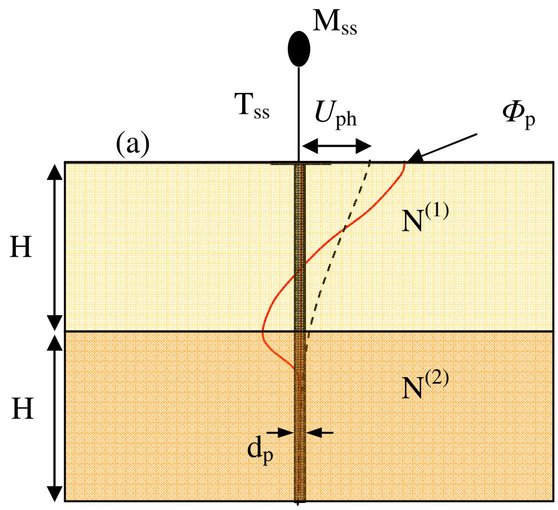
The determination of an optimal IM for prediction of a level of seismic demand is guided by the concepts of 'efficiency' and 'sufficiency', as defined by Shome and Cornell (1999). 'Efficiency' gives a measure of correlation of IM with EDP, and is typically measured via the standard deviation of the logarithm of the residuals, $\beta_{lnEDP|IM}$ (herein denoted simply as $\beta$). The residuals, $\varepsilon_i = y_i - y(x_i)$, represent the error between the raw data and some trend line (typically from regression). The better the efficiency of the IM, the smaller the value of $\beta$, which consequently reduces the number of analyses (i.e. EDP-IM data points) required to estimate the mean demand with a certain level of confidence.

The term 'sufficiency' refers to the independence of the residuals, $\varepsilon_i$, with respect to typical ground motion characteristics such as rupture magnitude and source distance. For example, if an IM is sufficient with respect to magnitude and distance, then effects of magnitude and distance can be ignored when predicting EDP without any loss in the accuracy of the prediction.

A suite of 40 ground motion records complied by Medina and Krawinkler (2003) were used for conducting the non-linear time history analyses. The suite contains ground motions recorded on stiff soil with magnitude and distance ranges of 6.5-6.9 and 13.3-39.3 km, respectively. The suite is termed 'ordinary' by Medina and Krawinkler, as none of the records show effects of near-fault motions (i.e. directivity or 'fling' effects), and all motions were recorded on stiff soils. This suite has been used by several other researchers (e.g. Tothong and Luco, 2007).

## ADOPTED SOIL-PILE-STRUCTURE MODEL

A conceptually simplified numerical model was used in this investigation, to try and capture the salient features of the pile response without onerous complexity. As shown schematically in Figure 1a the model consisted of a two-layer soil deposit with a single pile, rigid footing, and single degree-of-freedom (SDOF) superstructure. To investigate the effects of different material and geometrical properties, several different scenarios were considered (Figure 1b), which involved variations in the soil stiffness, pile properties, weight and period of the superstructure, based on typically observed configurations in engineering practice. Only the results of the first scenario is presented in this paper, an in-depth discussion of all six scenarios is given in Bradley et al, (2008). A finite element code was used which allows for direct effective stress analysis with pore water pressure development and dissipation. The Stress-Density (S-D) model of Cubrinovski and Ishihara (1998) was used as the constitutive model for the soil. The S-D model parameters were set such that the occurrence of liquefaction was suppressed in the scenarios considered herein. Effects of excess pore pressures and liquefaction were conducted in additional analyses which are beyond the scope of this paper. The shear modulus reduction curves for the soil were calibrated based on the well-known generalised curves of Seed and Idriss (1970) (Figure 1c). The pile and superstructure were modeled using beam elements with a hyperbolic moment-curvature relationship as an approximation to the 'exact' M-φ relationship of the 120 cm diameter RC pile, as shown in Figure 1d. The model was subjected to a base input motion scaled to peak ground accelerations between 0.1 and 1.0g in steps of 0.1g. Thus, using the 40 different ground motion records previously discussed a total of 400 analyses were performed for each of the scenarios listed in Figure 1b.




(b)

| Scenario | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| $N^{(1)}$ | 10 | 20 | 10 | 20 | 10 | 10 |
| $N^{(2)}$ | 30 | 30 | 30 | 30 | 30 | 30 |
| $W_{ss}$ (kN) | 2500 | 2500 | 400 | 400 | - | - |
| $T_{ss}$ (s) | 1.8 | 1.8 | 0.8 | 0.8 | - | - |
| $d_p$ (cm) | 120 | 120 | 40 | 40 | 120 | 40 |
| H (m) | 10 | 10 | 10 | 10 | 10 | 10 |

($N^{(i)}$=SPT blow count of layer *i*)

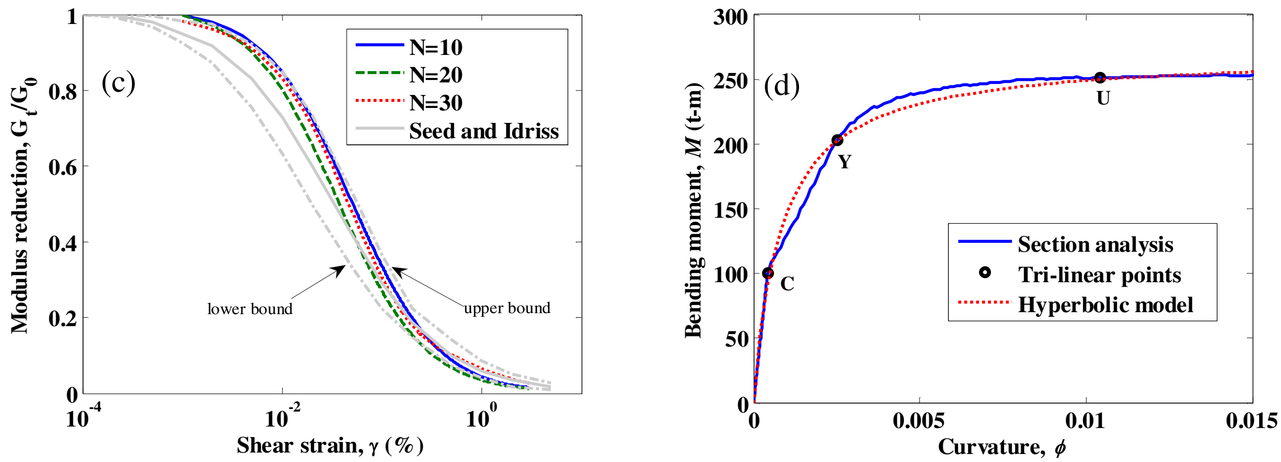



**FIG. 1. Soil-pile-structure model used in investigation: (a) schematic illustration of model; (b) scenarios considered; (c) modulus reduction curves; (d) hyperbolic approximation of M-$\Phi$ relationship for the pile.**

## MEASURE OF SEISMIC DEMAND ON PILES

The first question which must be asked when determining which ground motion intensity measures correlate well with seismic demand on piles, is how is the seismic demand measured? Ideally, the so-called engineering demand parameter (EDP) that is used would correlate perfectly with the occurrence of damage in the component. In comparison to the research attention that the EDP-IM relationship has received, little research has focused on determining optimal EDPs which correlate well with damage states (DS) in components. This is particularly true for foundations.

As with any engineering material, the seismic demand on a pile is generally related to the hysteretic energy released due to inelastic behaviour during ground shaking. Hysteretic energy is typically expressed as a function of both peak and cumulative deformations, one obvious example being the damage index of Park and Ang (1985). In the performance-based assessment of structural systems, typically cumulative plastic rotation and peak interstorey drift are used as the engineering demand parameters. When considering which EDP to use, consideration must also be given to the complexity in obtaining the EDP from engineering analyses. For example, while increasing numbers of finite element programs have the capability to compute damage indices (DIs), there are still some that do not, and also if simplified (i.e. not non-linear dynamic) analyses are to be used, then the computation of DIs is most likely not considered. For this reason peak response measures (e.g. peak floor acceleration, peak interstorey drift.) have been commonly adopted as the EDP for use in the fragility curve development of structural components (e.g. Tothong and Luco, 2007; Baker and Cornell, 2005). Following the same reasoning, only a peak response quantity will be used to quantify the seismic demand on the pile foundation in this study.

The peak pile curvature, $\Phi_p$, would seem the most obvious candidate to use for pile demand, as it directly provides the peak strain at the critical section of the pile and hence the damage level. However, in order to determine the peak pile curvature, the rotations at pile nodes throughout the length of the pile are required for each integration time step in the analysis. The curvature for each pile element is first computed and then the maximum must be identified along the length of the pile. It is not easily understood how this peak curvature relates to the global response of the pile. The peak lateral displacement of the pile head (i.e. at the foundation of the structure), $U_{ph}$, is such an EDP which is conceptually easier to understand. To make the EDP applicable to pile foundations of different geometry and soil stratification we will use a normalized peak lateral displacement of the pile head. The normalized peak lateral displacement of the pile head (herein referred to as pile head displacement for brevity) is defined as:

$$U_{ph}^{N} = \frac{U_{ph}}{h_{eff}} \tag{1}$$

where: $h_{eff}$ = effective height of the pile, which is the length between the location of the peak curvatures (potential plastic hinges) along the pile length. In this sense, $U_{ph}^{N}$ relates directly to inelastic rotation of the potential plastic hinges and therefore will correlate well with the occurrence of different damage states (e.g. cracking, yeilding, failure) in the pile. Note that as determination of the effective length of the pile is not the focus of this study, it has merely been taken as 10m (the depth of the upper soil layer) in further discussions. In other words, it has been assumed that the peak curvatures occur at the pile head and the interface between the soil layers. This is in agreement with observed damage to piles in past strong earthquakes.

It is intuitive that the peak pile head drift and the peak pile curvature are well correlated based on the likely first mode-dominated deformed shape of the pile during seismic excitation. Figure 2 illustrates the correlation between peak pile drift and peak pile curvature (for Scenario 1) from the 400 non-linear finite element analyses conducted in this study. The correlation indicates that $U_{ph}^{N}$ can be used to predict the deformation and hence damage of the pile.

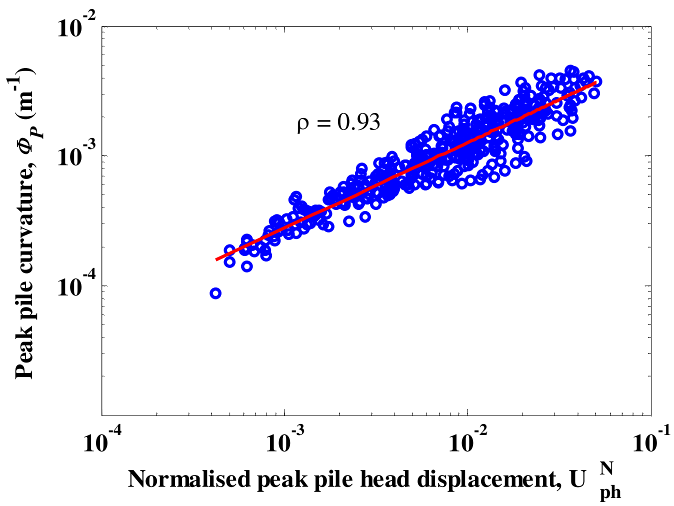



**FIG. 2: Correlation between normalized peak pile displacement and peak pile curvature from 400 nonlinear FE analyses for scenario 1**

## INTENSITY MEASURES INVESTIGATED

A total of 19 different candidate IMs were considered for correlation with the pile response (measured in terms of the normalized peak lateral pile head displacement), which are presented in Table 1. Definitions of all the IMs can be found in Riddell (2007). The list of candidate IMs cover acceleration-, velocity-, and displacement-based intensity measures, which include both peak and cumulative quantities. The periods of 0.4, 0.6, 0.8, and 1.8s which the spectral accelerations of IM 16-19 are computed at are selected to capture separately inertial and kinematic effects due to the vibration of the superstructure and imposed ground deformations, respectively.

**Table 1. Candidate IMs used in correlation analyses**

| ID | IM name | ID | IM name |
|---|---|---|---|
| 1 | Peak ground acceleration, PGA | 11 | Cumulative absolute velocity, CAV |
| 2 | Peak ground velocity, PGV | 12 | Acceleration spectrum intensity, ASI |
| 3 | Peak ground displacement, PGD | 13 | Velocity spectrum intensity, VSI |
| 4 | Significant duration, D | 14 | Sustained maximum acceleration, SMA |
| 5 | PGV/PGA, $V_{max}/A_{max}$ | 15 | Sustained maximum velocity, SMV |
| 6 | RMS acceleration, $RMS_a$ | 16 | Spectral acceleration, $S_a$(T=0.4s,5%) |
| 7 | RMS velocity, $RMS_v$ | 17 | Spectral acceleration, $S_a$ (T=0.6s,5%) |
| 8 | RMS displacement, $RMS_d$ | 18 | Spectral acceleration, $S_a$ (T=0.8s,5%) |
| 9 | Arias intensity, $I_a$ | 19 | Spectral acceleration, $S_a$ (T=1.8 s,5%) |
| 10 | Specific energy density, dE | | |

## RESULTS

### Efficiency

Firstly the efficiency of the candidate IMs was evaluated in an effort to reduce the number of IMs to focus on. For scenario 1 (see Figure 1b), Figure 3a depicts the efficiency of the candidate IMs (in-terms of $\beta$), with the numerical values of $\beta$ given in the third column of Table 2. Inspection of the results for scenario 2 showed similar trends as those for scenario 1, with values of $\beta$ typically varying by ±0.02. It becomes immediately apparent that the velocity-based IMs (e.g. PGV, $RMS_v$, SMV, VSI) all have good efficiency (smallest $\beta$ values) with respect to predicting the normalized peak pile displacement. The efficiency of the acceleration- and displacement-based IMs is noticeably less than the velocity-based IMs. Other IMs such as Arias Intensity ($\beta=0.402$) and *CAV* ($\beta=0.383$) are in the middle of the range of $\beta$ values. Also it is noted that while the efficiency of the spectral acceleration IMs (IM 16-19) is not poor, it is noticeably less efficient than the velocity-based IMs. As mentioned, Sa(1.8s,5%) was used to capture the inertial effects of the superstructure, and Sa(0.4-0.6s,5%) was used for the kinematic effects of the soil. As none of these IMs was able to efficiently (relative to the lower $\beta$ values observed) capture the pile response then it can be concluded that both inertial and kinematic effects are significant in this problem.

Over all of the six scenarios considered in this study it was found that VSI was consistently the most efficient IM, while the efficiency of PGV was dependent on the scenario considered (Bradley et al, 2008). As VSI is the integral of the velocity spectra (which is directly related to the acceleration spectra), then a ground motion prediction (attenuation) relationship (which describes the temporal occurrence of VSI), can be determined from ground motion prediction equations for spectral acceleration (Bradley et al, 2008). The availability of numerous attenuation equations for $S_a$ means that VSI will also be a predictable IM (i.e. a small scatter in the attenuation relation) (Kramer and Mitchell, 2006). For the above reasons we will use VSI as the optimal IM in further discussions, and we will also use PGA for comparison because of its conventional use as an IM. Figures 3b and 3c show the EDP-IM correlations for PGA and VSI, respectively. It should be noted that the reason the data points in Figure 3b are in 'strips' of constant IM is that the ground motion scaling factors for the records were based on scaling to PGA values. A significant difference in the scatter between the PGA- and VSI-based EDP-IM plots is clearly evident. A more rigorous evaluation of these IMs for a wider range of scenarios is given in Bradley et al, (2008).

### Sufficiency

As previously mentioned an IM should be sufficient (Shome and Cornell, 1998) with respect to rupture magnitude and source distance, in order to prevent bias when results are determined based on a finite ensemble of ground motions records. If an IM is sufficient with respect to a given ground motion parameter then it indicates that the results of the analysis are independent of that ground motion parameter. Since a finite ensemble of ground motion records is used to predict the response, if an IM is insufficient with respect a particular parameter then the response prediction will be sensitive to the distribution of this parameter in the ensemble of motions used.

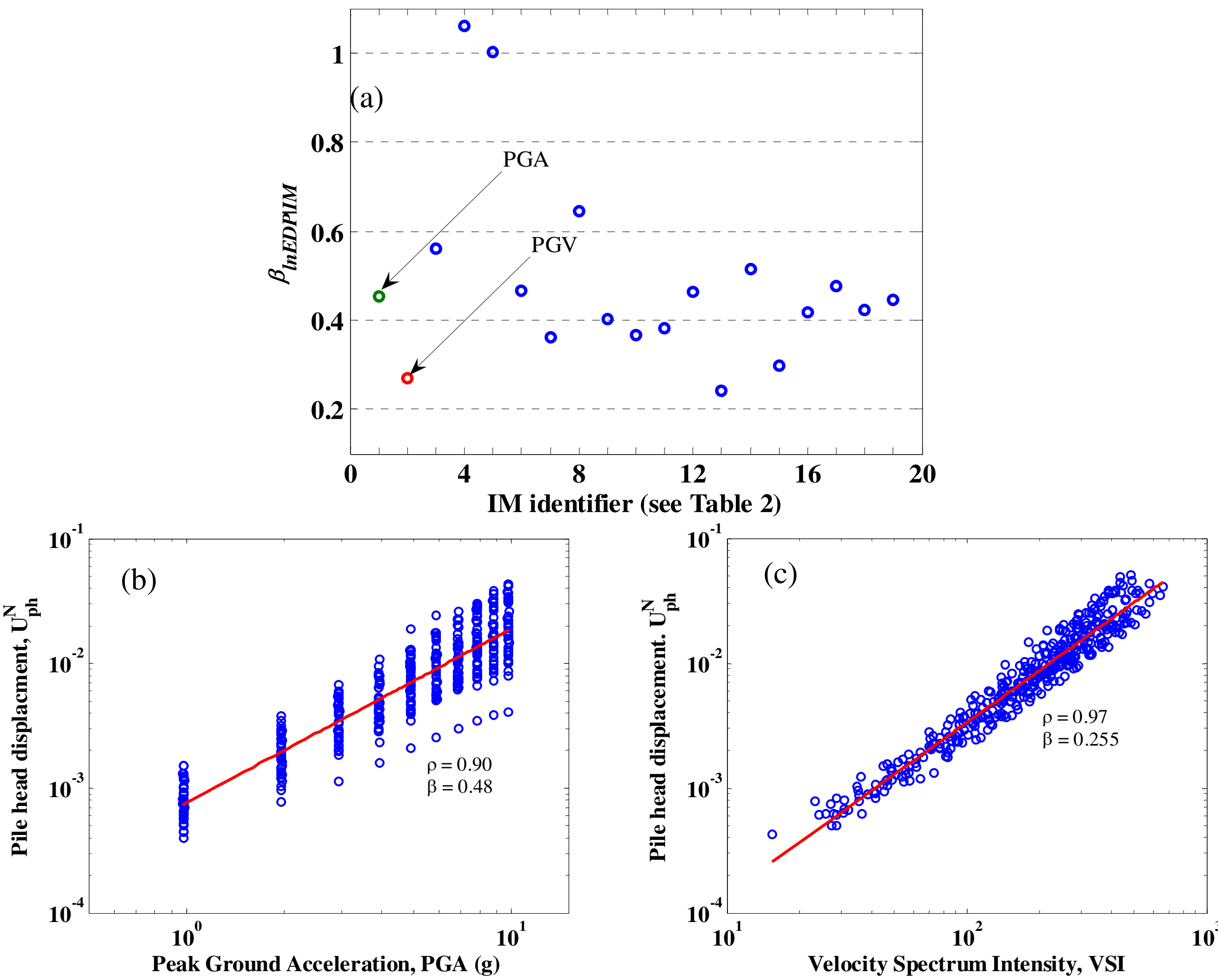



**FIG. 3: Efficiency of the candidate IMs. (a) Efficiency of all IMs as a function of $\beta_{lnEDP|IM}$; (b)&(c) EDP-IM scatter plots for PGA and VSI, respectively.**

In order to determine if an IM is sufficient with respect to a ground motion parameter, first the residuals, $\varepsilon_i$, have to be computed for the respective IMs. Here, the residual represents the error in the prediction by the regression model with respect to the actually computed EDP. Therefore, based on the previous statements if an IM is sufficient with respect to a given ground motion parameter there should be no trend in the residuals as a function of the ground motion parameter, that is, the residuals should be statistically independent of the ground motion parameter. Such independence is typically quantified by determining the 'p-value' (Ang and Tang, 1975), which corresponds to the probability that the slope, b, of the regression line through the epsilon-ground motion parameter is equal to zero. Typically if the p-value is less than 0.05 then there is evidence to reject the null hypothesis that the slope (b) of the regression line is zero. If $p < 0.01$ then it is said there is significant evidence to reject the null hypothesis.

**Table 2. Efficiency and Sufficiency of IMs from analysis (scenario 1)**

| | | | Magnitude | | Distance | | Scale factor | |
|---|---|---|---|---|---|---|---|---|
| ***IM*** | $\rho$ | $\beta_{\ln EDP\|IM}$ | ***b*** | ***p*-value** | ***b*** | ***p*-value** | ***b*** | ***p*-value** |
| 1 | 0.898 | 0.475 | 0.66 | 0 | -0.19 | 0.007 | 0.05 | 0.052 |
| 2 | 0.97 | 0.261 | 0.25 | 0.010 | 0.07 | 0.062 | 0.02 | 0.244 |
| 3 | 0.864 | 0.542 | 0.98 | 0 | 0.4 | 0 | 0.21 | 0 |
| 4 | 0.168 | 1.062 | 0.9 | 0.013 | -0.2 | 0.209 | 0.93 | 0 |
| 5 | 0.38 | 0.997 | 0.36 | 0.291 | 0.03 | 0.859 | 0.89 | 0 |
| 6 | 0.891 | 0.488 | 0.81 | 0 | -0.17 | 0.023 | 0.01 | 0.671 |
| 7 | 0.946 | 0.35 | 0.75 | 0 | 0.2 | 0 | 0.03 | 0.198 |
| 8 | 0.812 | 0.628 | 1.4 | 0 | 0.35 | 0 | 0.25 | 0 |
| 9 | 0.919 | 0.424 | 0.64 | 0 | -0.14 | 0.029 | -0.01 | 0.596 |
| 10 | 0.945 | 0.351 | 0.6 | 0 | 0.21 | 0 | 0.05 | 0.011 |
| 11 | 0.929 | 0.399 | 0.75 | 0 | -0.11 | 0.071 | -0.03 | 0.2 |
| 12 | 0.892 | 0.486 | 0.64 | 0 | -0.17 | 0.018 | 0.05 | 0.092 |
| 13 | 0.972 | 0.255 | -0.05 | 0.565 | 0.02 | 0.546 | 0.04 | 0.003 |
| 14 | 0.866 | 0.539 | 0.47 | 0.011 | -0.22 | 0.006 | 0.07 | 0.017 |
| 15 | 0.962 | 0.294 | 0.24 | 0.017 | 0.13 | 0.003 | -0.01 | 0.719 |
| 16 | 0.911 | 0.443 | 0.33 | 0.031 | -0.2 | 0.003 | 0.07 | 0.008 |
| 17 | 0.886 | 0.499 | -0.45 | 0.008 | -0.23 | 0.002 | 0.25 | 0 |
| 18 | 0.913 | 0.439 | -0.73 | 0 | -0.3 | 0 | 0.19 | 0 |
| 19 | 0.918 | 0.427 | 0.28 | 0.057 | 0.09 | 0.183 | 0.13 | 0 |

Figures 4a and 4b give the magnitude sufficiency plots for VSI and PGA, respectively. It can be seen that the dependence of magnitude on the PGA-based residuals is significantly larger than that for VSI. It should be noted that some care should be taken when viewing Figure 4 since the magnitude distribution of the as-recorded ground motions is very coarse. Figure 4a indicates that the b-value (the slope of the regression line) is 0.25 and a corresponding probability of 56% that the slope of the trend line is zero. Therefore it can be stated that the VSI-based prediction is independent of magnitude (over the limited range considered).

Figures 4c and 4d illustrate the sufficiency of VSI and PGA with respect to source distance. As for sufficiency based on magnitude, the dependence of the VSI-based prediction on source distance is less than that for the PGA-based prediction. Hence, in addition to a better efficiency, VSI is also more sufficient than PGA with respect to rupture magnitude and source distance.

## CONCLUSIONS

From the results of this investigation the following conclusions can be drawn:

1. Good correlation between the normalized peak lateral displacement of the pile head and the peak curvature of the pile was obtained from a series of 400 non-linear FE analyses indicating the use of the peak pile displacement as a rational EDP for piles under lateral loading
2. By considering a simple soil-pile-structure model it was found that for the scenarios investigated in this study, velocity-based measures of intensity (such as velocity spectrum intensity, VSI) correlate best with the seismic demand on pile foundations.
3. VSI was found to be more efficient than PGA and was also sufficient

(independent) with respect to rupture magnitude and source distance.

The model used in this study considered only a single pile and a SDOF supersturucture. Therefore, pile group effects and higher mode effects were ignored in this study. Also, the IMs considered were all of a scalar form. If use of a vector-based IM may improve the prediction of the pile response (EDP) through more sophisticated treatment of the inertial and kinematic effects of the pile response.

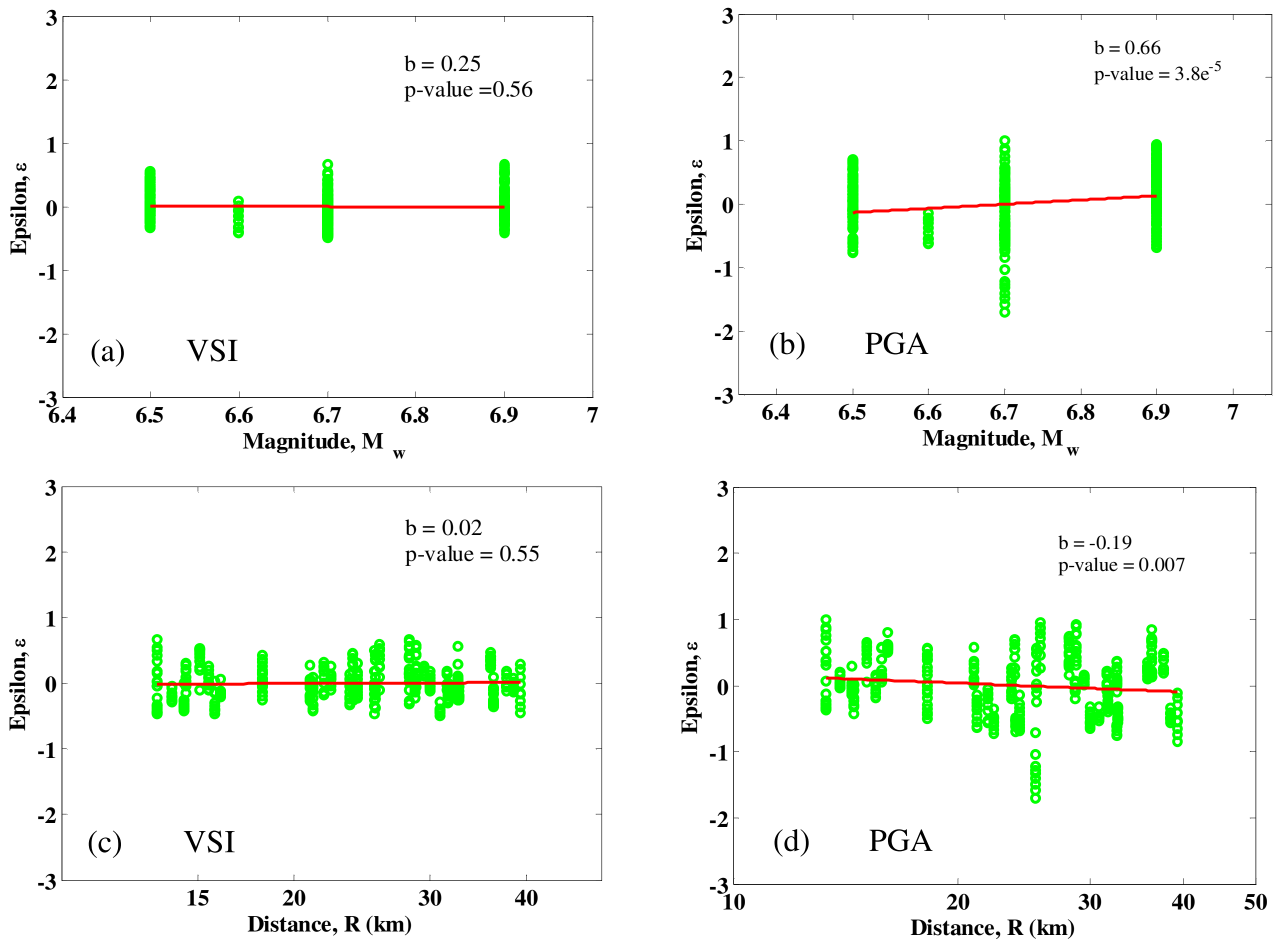



**FIG. 4: Sufficiency of velocity spectrum intensity and peak ground acceleration with respect to magnitude and distance**

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