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Seismic Performance of Piles from PBEE and EQWEAP Analyses

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ABSTRACT: The Performance Based Design in geotechnical engineering requires an extensive research work prior to the details establishment for the design. The seismic performance of the piles is certainly of this interest, thus worthwhile discussions for the engineers. This paper would allow one to find the example of the studies based on PBEE and EQWEAP analyse. A numerical study was conducted for the piles located in Taipei Basin where the seismic conditions are significantly important to the design engineers. Therefore the local seismic design concerns of the Building Code were also incorporated into the measurements. Follow the simplified form of PEER Framing equation, probabilities of the possible pile performance parameters were able to examine whereas the prospectives of such analyses are suggested accordingly.

Keywords: Seismicity, Performance Based Earthquake Engineering, Wave Equation Analysis, Pile Foundation

1. INTRODUCTION

Performance based design has received tremendous attentions from geotech societies in recent years. GeoCode-21, and Eurocode 7 were both developed for PBD concerns. To estimate the seismic performance of the structures, the so called *Framing equation* was suggested by US PEER for performance based earthquake engineering (PBEE) analysis. In such anysis, the probability in terms of the annual rate of exceedance for the intensity measure (IM) of the earthquake, the engineering demand parameter (EDP) and the damage measure (DM) of the structure as well as the decision variable (DV) can be evaluated for the structural design, and the corresponding decisions in the managements can be analyzed using step-by-step discrete procedures. One can estimate the probabilities of structural parameters and compare them to the limited values for design purpose. For a pile foundation located at a site with known ground conditions, the seismic displacements and internal moments of the piles could be measured for many possible earthquake excitations. One can estimate the probabilities of these quantities following the PBEE procedures, and the performance of the structure can be estimated with all possible influence factors. By proper controls of the factors, the analysis is an applicable tool to evaluate the seismic performance of the earth structures. For analysis for structural behaviors, static and pseudo static analysis as well as the dynamic analysis are all available tools. In this paper, the wave equations of the pile segments subjected to the seismic ground motions are suggested for simplicity and time-dependent capability. A so called EQWEAP analysis is adopted for analysis of the piles. The design practice for pile foundations and concurrent PBD concerns in Taiwan are introduced with a numerical example to show the applications of these analyses.

2. PBEE ANALYSIS

Comprehensive overview of the PBEE analysis can be found in Kramer (2008). The ground motions, structural responses, physical damages and loss should be carefully analyzed considering the occurrence of the influence factors and the reliability of the design factors of interest. The IM, EDP, DM and DV values are to be analyzed accordingly. The *Framing* equation proposed by PEER is written as follows,

$$\lambda(DV) = \iiint \begin{matrix} G(DV \mid DM) \mid dG(DM \mid EDP) \mid\mid \\ dG(EDP \mid IM) \mid\mid d\lambda(IM) \mid \end{matrix} \tag{1}$$

In Eq. (1), $G(a|b)$ denotes a complementary cumulative distribution function (CCDF) for a conditioned upon b (the absolute value of the derivative of which is the probability density function for a continuous random variable). The three CCDFs result from the loss, damage, and response models; the final term, $d\lambda(IM)$ is from the seismic hazard curve. This triple integral can be solved numerically for most practical problems as follows.

$$\begin{aligned} \lambda_{DV}(DV) = \sum_{k=1}^{N_{DM}} \sum_{j=1}^{N_{EDP}} \sum_{i=1}^{N_{IM}} & P[DV > dv \mid DM = dm_k] \\ & P[DM > dm_k \mid EDP = edp_j] \\ & P[EDP > edp_j \mid IM = im_i] \Delta\lambda_{IM}(im_i) \end{aligned} \tag{2}$$

The numerical integration can be accomplished as where $P[a|b]$ describes the probability of a given b, and where N_{DM}, N_{EDP}, and N_{IM} are the number of increments. According to Kramer (2008), the discrete form shown in Eq. (2) can be broken down into a series of components. The individual conditional probability terms can be expressed in the form of fragility curves. With some simplifying assumptions, the *Framing* equation can be solved in a closed form with the use of a power law relationship between mean annual rate of exceedance and *IM*.

$$\lambda_{IM}(im) = k_0 (IM)^{-k} \tag{3}$$

In Eq. (3), k_0 is the value of λ_{IM} (im = 1) and k is the slope of the seismic hazard curve. If the response model is also assumed to be of power law form, then

$$EDP = a(IM)^b \tag{4}$$

Based on lognormal dispersion that has statistically independent aleatory and epistemic components of uncertainty β, the *EDP* hazard curve can be expressed as

$$\lambda_{EDP}(edp) = k_0 \left[\left(\frac{edp}{a} \right)^{1/b} \right]^{-k} \exp\left[\frac{k^2}{2b^2} \left(\beta^2 \right) \right] \tag{5}$$

Eq. (5) describes the mean annual rate of exceeding some level of response, $EDP = edp$, given the seismic hazard curve and a probabilistic response model. One could find detailed explanations regarding the use of this equation and corresponding ones when the damage and loss models were involved in Kramer (2008). For response model in use, the numerical solution of the
annual rate of exceedance, λ□ for a certain level of *edp* can be expressed as:

$$\lambda_{EDP}(edp) = \sum_{i=1}^{N_{IM}} P[EDP > edp \mid IM = im_i] \Delta\lambda_{IM}(im_i) \tag{6}$$

If one would like to omit the hazard rate increments used for the integrations, simplified procedures to obtain the "stripes" data and the "cloud" data can be used to find out the simple relationships of EDP and IM, and the results shall then become much easier to obtain (Kramer, 2008). In using this procedure to analyze the bridge pier foundation, Shin (2007) found that the uncertainty of the earthquake is mostly significant to the analysis. More than 80% uncertainties will resolved from this variable. Sometimes, the effects of the soil parameters and the geological profiles were studied too. The details could be found in Shin's dissertation (2007). It is necessary to point out that any proper structural analysis can be incorporated with the PBEE procedures for the estimations.

3. EQWEAP ANALYSIS

Seismic responses of the piles could be analysed using the time-dependent Winkler type foundation model, whereas a simplified two-step procedure EQWEAP was suggested by the author (Chang *et al.* 2001 and 2003). The free-field ground motions are obtained first and then applied to the pile for the solutions, and the discrete wave equations are used to solve for the pile displacements. Figure 1 illustrates the layout of the procedure. This modelling was reported in good agreements with the FE ones. To model the soil liquefaction and/or liquefaction induced lateral spreading, a number of alternative models have been suggested (2006, 2007[a,b], 2008[a,b,c]).

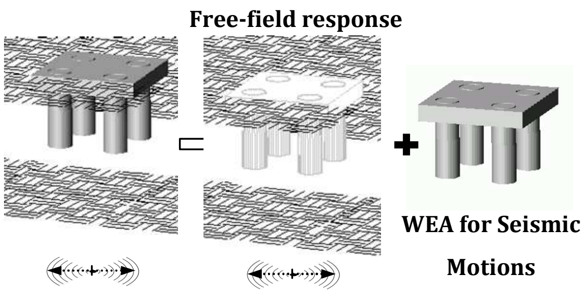

Figure 1. Uncoupled procedures used in EQWEAP

The 1st step in EQWEAP procedure simply adopts lumped mass model to solve for the free-field ground responses. It is rather convenient and simple analysis, nevertheless one must be cautioned to conduct the analysis using the bedrock accelerations and the base line corrections of the displacements. In the 2nd step, the resolved ground motions should be applied to the springs and dashpots along the pile in order to calculate the seismic forces applied to the pile. The effects of pile-to-pile interactions and the soil-cap-pile interactions can be further included. If the seismic earth pressures were known beforehand, then the 1st step analysis can be omitted. On the other hand, if the seismic ground motions were prescribed already and the subgrade reaction modulus of the soils could be used to model for the soil impedances, the corresponding earth pressures could be computed and applied to the pile for the solutions. All these are feasible solutions to model the seismic pile responses.

For solution of the liquefied soil, the soil parameter reduction coefficients suggested by the Japan Road Association (JRA, 1996) could be considered. The reduction coefficients are applied to both the free-field ground response analysis and the wave equation analysis to reduce the soil stiffness due liquefaction. This approach is rather simple but rational enough to reproduce the degraded modulus of liquefied soil.

One can also use the excess pore-water pressures (PWP) model to simulate the liquefaction. For example, Finn's model (1977) has been adopted by the authors to simulate the liquefaction influences on pile. The volumetric strains of the soils were computed and accumulated during the seismic excitations to obtain the excess pore water pressures. Soil liquefaction is modeled through the ground response analysis. Shear modulus of the soil could be calibrated using the suggestion of Seed and Idriss (1970) or any similar ones with the dependence of shear strains.

Iterative procedures were performed to ensure the convergence and equilibrium of the structural system. This approach waives the conduction of liquefaction potential analysis. For solutions adopting the direct earth pressures and the indirect ones from the empirical ground displacement profiles, a more recent study of the author (2008[c]) can be referred. It has been reported that the EQWEAP can provide good estimations for seismic pile responses to an extent that the pile damages may occur.

For large earthquake excitations, nonlinear pile behaviors based on the moment-curvature relationships were considered. It can be obtained from both experiments and rigorous computations. For simplicity, one can approximate the nonlinear curves with the bi-linear (steel pipe piles) or tri-linear (concrete piles) relations. For given values of the cutting-point moments and their corresponding curvatures, one can find the approximate model constants for each line. With the EI values adjusted iteratively through the wave equation analysis, one can obtain approximately the nonlinear pile responses. The details of EQWEAP can be found in a recent paper summarized by the authors (2010). Assuming fixed head and long pile conditions, the basic forms of the solutions of EQWEAP can be derived as follows.

General formulation:

$$u_p(i, j+1) = \frac{1}{C_1 + C_3}\begin{bmatrix} -u_p(i+2, j) + \\ (4 - C_2)u_p(i+1, j) \\ -(6 - 2C_1 - 2C_2 + C_4)u_p(i, j) \\ +(4 - C_2)u_p(i-1, j) \\ -u_p(i-2, j) - \\ (C_1 - C_3)u_p(i, j-1) \\ +C_3[u_s(i, j+1) - u_s(i, j-1)] \\ +C_4 u_s(i, j) \end{bmatrix} \quad (7)$$

Notethat $C_1 = A\Delta z^4 / V_c^2 I \Delta t^2$, $C_2 = P_x \Delta z^2 / EI$, $C_3 = C_s \Delta z^4 / 2\Delta t EI$, $C_4 = K_s \Delta z^4 / EI$.

In above equation, i is the *ith* nodal point, j is the *jth* time step, V_c is the compressive wave velocity of the pile, and is equal to $(E/\rho)^{1/2}$, Δz and Δt are the thickness of the pile segment and time increment respectively, E=Young's modulus of pile, I= moment of inertia of pile, ρ=mass density of pile, A=cross-section area of pile, P_x=vertical load, u_p=absolute pile displacement, u_s= absolute soil displacement, u= u_p-u_s=relative pile displacement, C_s and K_s=damping coefficient and spring constant of the soils along the pile. Following equations are the ones derived considering boundary conditions:

Equation for the pile head:

$$u_p(i, j+1) = \frac{1}{C_1 + C_3}\begin{bmatrix} -2u_p(i+2, j) \\ +(8 - 2C_2)u_p(i+1, j) \\ -(6 - 2C_1 - 2C_2 + C_4)u_p(i, j) - \\ (C_1 - C_3)u_p(i, j+1) \\ +C_3[u_s(i, j+1) - u_s(i, j-1)] \\ +C_4 u_s(i, j) + C_6 \end{bmatrix} \quad (8)$$

where $C_6 = 2\Delta z^3 P_t / EI$, P_t = horizontal load at the pile head.

Equation for node right beneath the pile head:

$$u_p(i, j+1) = \frac{1}{C_1 + C_3}\begin{bmatrix} -2u_p(i+2, j) + \\ (4 - C_2)u_p(i+1, j) \\ -(6 - 2C_1 - 2C_2 + C_4) \\ u_p(i, j) + (4 - C_2)u_p(i-1, j) \\ -(C_1 - C_3)u_p(i, j-1) + C_3 \\ [u_s(i, j+1) - u_s(i, j-1)] \\ +C_4 u_s(i, j) + C_6 \end{bmatrix} \quad (9)$$

Equation for the pile tip:

$$u_p(i, j+1) = \frac{1}{C_1 + C_3} \left[\begin{array}{l} (2C_1 + 2C_2 - C_4 - 2)u_p(i, j) \\ +(4 - 2C_2)u_p(i-1, j) \\ -2u_p(i-2, j) \\ -(C_1 - C_3)u_p(i, j-1) \\ +C_3[u_s(i, j+1) - u_s(i, j-1)] \\ +C_4 u_s(i, j) \end{array} \right] \quad (10)$$

Equation for node right above the pile head:

$$u_p(i, j+1) = \frac{1}{C_1 + C_3} \left[\begin{array}{l} (2 - C_2)u_p(i+1, j) \\ -(5 - 2C_1 - 2C_2 + C_4)u_p(i, j) \\ +(4 - C_2)u_p(i-1, j) \\ -u_p(i-2, j) - (C_1 - C_3)u_p(i, j-1) \\ +C_3[u_s(i, j+1) - u_s(i, j-1)] \\ +C_4 u_s(i, j) \end{array} \right] \quad (11)$$

4. PILE DESIGN PRACTICE IN TAIWAN

A number of design codes are available for pile foundation design in Taiwan. All the design codes and specifications require the checks for foundation capacities at ordinary and seismic conditions. The settlements and deformations of the foundation also need inspections. In general, both working stress design (WSD) and limit state design (LSD) are adopted in current design practice. Fig. 2 shows the flowchart of a generalized pile design procedures taken in Taiwan. It can be seen that the seismic concerns were mainly focusing on the foundation capacities, whereas the liquefaction effects are considered independently. The flow pressure model for liquefaction-induced lateral spreads and the soil parameter reduction coefficients from liquefaction potential analysis of the site (JRA, 1996) were mainly used. The pile design details and the notes on procedures taken in different aspects were summarized by Chang *et al*. (2008) as an in-house publication of MAA, Inc.

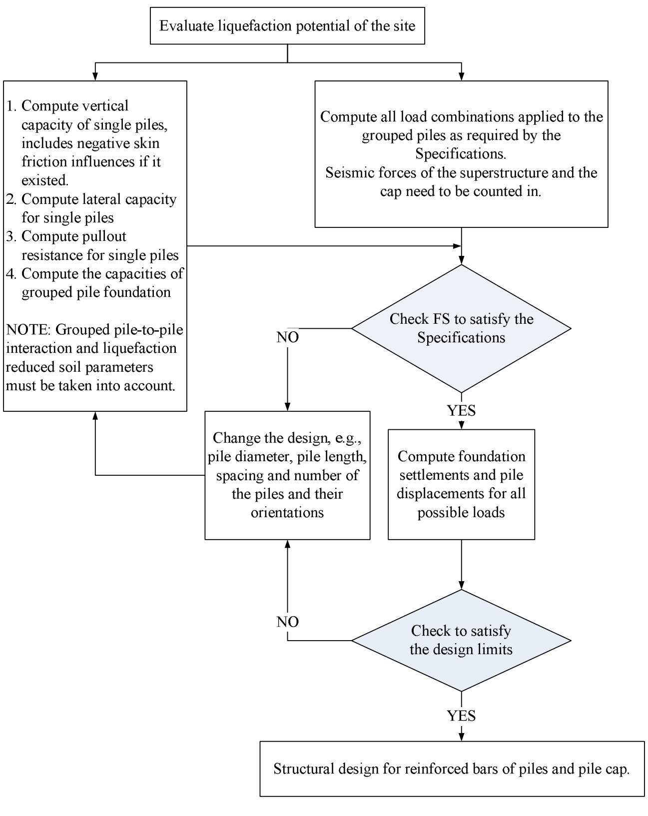

Figure 2. Common pile design procedures undertaken in Taiwan

4.1 PBD Work under Developments

The development of a new geotechnical design code has been initiated at Taiwan Geotechnical Society (TGS) in the past years. The load and resistance factor design (LRFD) and performance based design (PBD) have been introduced to local engineers since 2000. The relevant works started to boom after *The 2nd Int. Symposium on New Generation Design Codes for Geotech. Engr. Practice* held in Taipei in 2006. A number of international scholars have demonstrated their experiences on this issue. Reliability analysis for the design and the performance of the structures in lines with the limit state design for the elements and the members were discussed. Accordingly the seismic performance of the geotechnical structures is receiving many attentions. In the meantime, Chen *et al.* (2006) introduced the Design concepts for Seismic Performance of the Pile Foundations for Bridge Piers to TGS. According to their suggestions, the seismic performances of the pile foundations could be categorized into three levels with the concerns of foundation serviceability, rehabilitation and safety, respectively (see Tables 1 to 3).

Performance Level I indicates that the structure is mainly governed by the elastic behaviors under small to medium earthquakes, where soil liquefaction does not occur or occurs slightly. The major interest of Level I is the serviceability of the structure. Conventional design methods are applicable in this case. Performance Level II is applicable to medium to large earthquakes, nonlinear structural responses can be resulted, in which the ground tends to liquefy. The major concern of Level II is the rehabilitation and safety of the structure, both short term and long term should be evaluated. The engineers need to make sure that any local damage of the structure is not allowed in this case. Performance Level III is amendable to nonlinear responses of the structures that are affected by soil liquefaction and liquefaction induced lateral spread of the ground under very large earthquakes. The fatal collapse of the structure is prohibited in this requirement. Notice that the relationships between theses performance levels and the return periods of 30, 475 and 2500 years are referable in Table 2.

Chen *et al.* (2006) also suggested that nonlinear static and dynamic analyses could be applied based on complexity of the problem. For design practice following their suggestions, the alternate approaches (see Fig. 3) are suggested herein. The approaches for liquefaction and liquefaction induced lateral spreading can be considered using conventional static Winkler foundation model and/or the dynamic one (e.g. FEM or EQWEAP). In applying EQWEAP with different models and comparing the dynamic solutions with the static ones for a number of case studies, Chang *et al.* (2006, 2007[a,b] and 2008[a,b,c]) had shown that the dynamic and static solutions are agreeable to a certain extent.

4.2 Analysis and Design with Seismic Concerns

There are a number works in demonstrating validities of these models using different techniques. For example, Winkler's foundation model was suggested by Hwang (2000). A pesudo static solution was suggested by Lin *et al.* (2005) applying the uncoupled analysis to model the pile damage under lateral spread. Hwang and Chung (2006) lately suggested a simplified closed form solution for piles subjected to liquefaction induced flow pressures. Chang *et al.* (2003, 2006) on the other hand have successfully incorporated these models into the EQWEAP procedures for dynamic pile responses due the earthquake excitations. Simplified moment-curvature relationships of the piles are generally used in these studies to model the nonlinear pile responses. A few other studies using linear/nonlinear finite element analyses could be found. However, due to the complexities of the modeling and the material laws, the FE analysis is seldom used in routine designs. This rigorous analysis is only applied to certain projects, in which the macroscopic influences of the structures, the geographic conditions and the geological data need to be considered carefully.

Table 1. Seismic Performance Concerns for Transportation Structures (after Chen *et al.*, 2006)

Performance	Safety	Serviceability	Rehabilitation	
			Short term	Long term
Level I	structure remains elastic	same as before	not needed	routine monitoring, protections
Level II	restricted local damages, recoverable	recoverable w/ short-term remedies	urgent remedy method applicable	existing remedy method applicable
Level III	superstructure and main body collapse prohibited	urgent remedies applicable, limited speed/weight for vehicles	Replacing elements, structural reinforcements undertaken	closed for reconstructions

Table 2. Seismic Performances and Return Periods for Transportation Structures (after Chen *et al.*, 2006)

Hazard Level	Embankment	Bridge pile foundation		Underground structures	
		ordinary	important	ordinary	important
S_{30}	Level I	Level I		Level I	
S_{475}	Level III	Level III	Level II	Level III	Level II
S_{2500}	N/A	N/A	Level III	N/A	Level III

1. Level I: elastic deformations, no or rare liquefaction, in normal condition.
2. Level II: plastic deformations allowed, slight to medium liquefaction, recoverable damages.
3. Level III: ultimate deformations occurred, severe soil liquefaction, structure not allow to collapse.

Table 3. Analyses for Seismic Performances of Transportation Structures (after Chen *et al.*, 2006)

Performance	Soil and structural behaviors relatively simple	Soil and structural behaviors relatively complicated
Level I	Linear static analysis	Linear static analysis
Level II	Nonlinear static analysis	Nonlinear static analysis or Nonlinear dynamic analysis
Level III	Nonlinear static analysis	

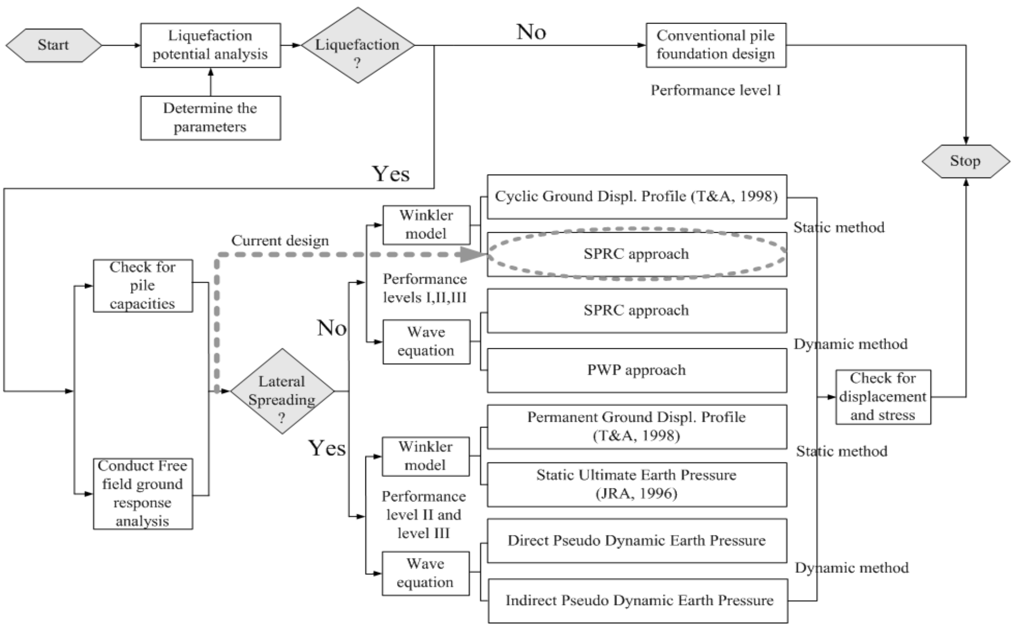

Figure 3. Seismic performance-based design procedures and methods suggested for pile design in Taiwan

At this time being, no standard package is used for pile foundation design in Taiwan. As long as the tool can satisfy the required specifications, it is a valid design tool. For example, computer programs APILE, LPILE and GROUP have been adopted by several major firms. Some private sectors use SAP2000 to analyze the pile behaviors. All these commercial packages could provide rational information for further structural designs. As to the practice, the liquefaction and liquefaction-induced lateral spread influences are analyzed step by step in the design. In general, the liquefaction potential of the site is checked first. If soil liquefaction occurs, the soil parameter reduction coefficients can be used to reduce the spring constants and the strengths of the soils. Liquefaction effects are simulated in subsequent analyses for capacities and deformations of the foundation. The reduced foundation capacity is thus compared to the specified load combinations at ordinary time and seismic condition to ensure the safety. The most critical loads are used for structural design of the cap and the piles. Again, maximum displacements of the foundation were computed and comparing with the limits.

If the lateral spread is a major concern, then the flow pressures were used to model the pile displacements. Pile damages are examined. Accordingly, the method selected for liquefaction potential analysis and the design seismicity are important to the results. The JRA method (1996), T&Y method (Tokimatsu and Yoshimi, 1983) and the NCEER method (or modified Seed method, 1997) are often adopted by local engineers to evaluate the liquefaction potential of the site. The seismic design code for buildings in Taiwan has been modified after the 1999 Chi-Chi earthquake. Figure 4 illustrates the old version of the seismic zones suggested. Note that PGAs of 0.33g and 0.23g are respectively suggested for zone 1 and 2 in Taiwan after Chi-Chi earthquake. The corresponding design earthquake is designated with a 475 year return period. In 2006, the newest seismic design code for buildings was released. It follows the updated procedures suggested in International Building Code. Again, three target earthquakes with return period of 30, 475 and 2500 years were considered for earthquakes at different levels. The corresponding PGA values at various districts and cities in Taiwan were respectively suggested for short period (0.3sec) and medium long period (1sec) structures. The ground stiffness and fault distance are considered to modify the design PGA.

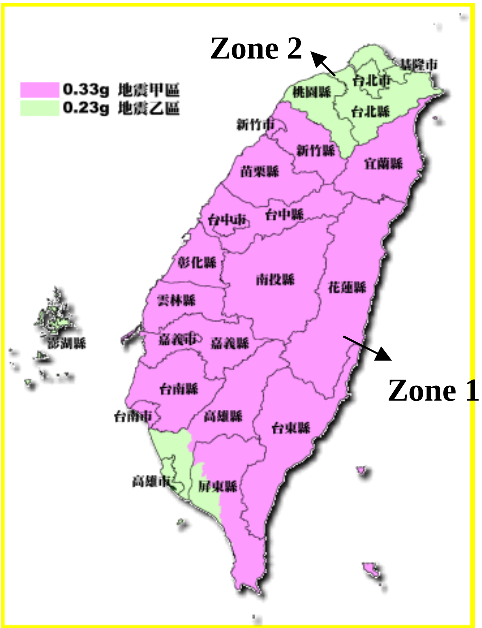

Figure 4. Seismic zones suggested in old seismic design code in Taiwan

5. SEIMICITY IN TAIWAN

The importance of the fault sources and closest distance to fault in developing the ground-motion attenuation relationships is pronounced in PSHA. Cheng (2002) has successfully used the logic tree and weightings at branches to discuss the uncertainty of PSHA considering the earthquakes in Taiwan. The characteristics of seismic sources in vicinities by deaggregating hazard contributed from different magnitude and distance were carefully examined. 3-D plate source to model fault planes and subduction zone plates was used besides the regional sources. Truncated-Exponential model developed by mainshock of EQ in M_W from 1900 to 1999 was used to describe the magnitude distribution of regional sources. Characteristic-Earthquake model developed by fault slip rate was used to describe the magnitude distribution of active fault and subduction interface sources.

Adopting suitable attenuation relationship for each source in PSHA, especially the crustal source including the Chi-Chi earthquake sequence, the hanging-wall effect and site condition for specific site was revealed. According to the iso-seismic hazard map of PGA, 0.2sec and 1.0sec spectral acceleration, the hazard level is strongly dependent of the fault. The hazard was found significant around the centre of the hanging wall. The highest hazard level can be found in the eastern longitudinal valley and western foothills to coast plain, separated by the central mountain range in low hazard level. Furthermore, the hazard level considering faults activity divided by regional sources shows that the prominent ratio always distribute on hanging wall. This occurs especially on the low background seismicity region such as Taichung, Hsinchu and Miali. Figure 5 presents the hazard crves read from Cheng's study (2002) for Taipei, Taichung and Kaoshiung cities. From the deaggregation of PSHA, Cheng was able to show that the hazard contributed mainly from the distance and magnitude bin by different return period. The deaggregation process could provide information for hazard mitigation while choosing scenario earthquakes. Of course, there are some other representable hazard curve results in Taiwan. For example, the ones proposed by NCREE (2002) were suggested based on characteristic earthquakes.

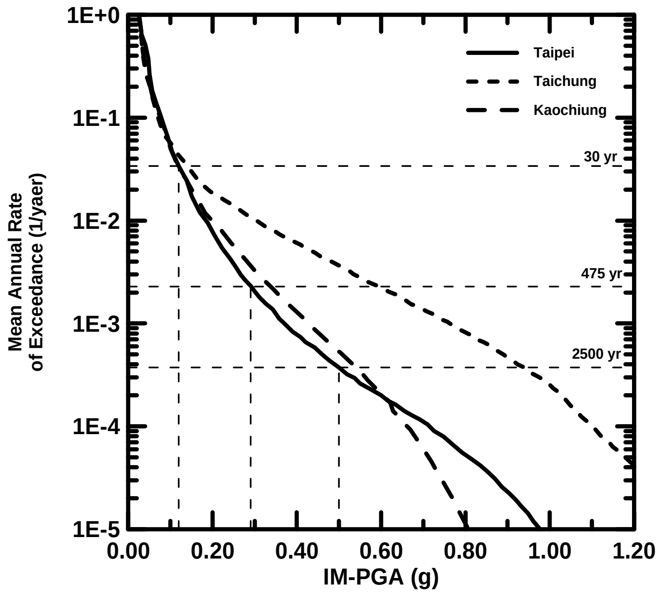

Figure. 5 Hazard curves of Taipei, Taichung and Kaoshiung (based on total mean curves by Cheng, 2002)

The data bank and number of sites considered in the seismic hazard study will result different results. One must be cautioned when using the hazard curves to conduct the analyses. For seismic design code used currently in Taiwan, the structures can be designed at three levels of seismic resistances to accommodate the ordinary EQs, the design EQs and the maximum considered EQs. The return periods of these earthquakes for a time of 50 years with the occurrence probability of 80%, 10% and 2% can be found as 30, 475 and 2500 years, respectively. The mean annual rate of exceedance is simply the reciprocal of the return period.

6. EXAMPLE STUDY

In Fig. 5, the associated PGAs at return period of 30, 475 and 2500 years in Taipei are 0.12, 0.29 and 0.51*g*, respectively. For simplicity, the regression analysis of the PGAs can show that the hazard model can be expressed as a power function with k=3.071 and k_0=4.917E-5 (r^2=0.995). Using the simplified procedure as the stripes data from the response model, the above PGAs are taken as the target PGAs for response analysis of a single pile installed in Taipei. According to the available earthquake data and seismic records as well as the site information, the authors select the accelergrams recorded at 6 seismic stations in Taipei Basin considering 1999 Chi-Chi earthquake (in-land, active faulting triggered quake) and 2002 Yi-Lang earthquake (east coast offshore, subduction plate triggered quake). Figure 6 illustrates the earthquake records in use. Only the maximum horizontal ground excitations are considered for the analysis. The geological data of the sites were found very similar, in which the averaged shear wave velocity of the soils at the depths of upper 30m of the ground is approximate 200m/sec.

Figures 7 and 8 illustrate the locations and the velocity profiles of these stations, whereas Fig. 9 shows the ground profile of the foundation site. Typical pile dimensions (length=29*m*, diameter=1*m*) and stiffness properties ($EI=1.2\times10^{6} kN\text{-}m$) are assigned for pile response analysis. The EQWEAP analysis with the Finn's PWP model were conducted to obtain the dynamic pile responses subjected to these earthquake excitations. The maximum pile displacement occurring at the pile head (with restraints against rotation) is taken as the EDP. Figure 10 presents the demand curves obtained using the medians of the discrete data for the target PGAs (IM values). If a power law is used, the corresponding *a* and *b* parameter will be 488.0 and 1.563 (r^2=0.998).

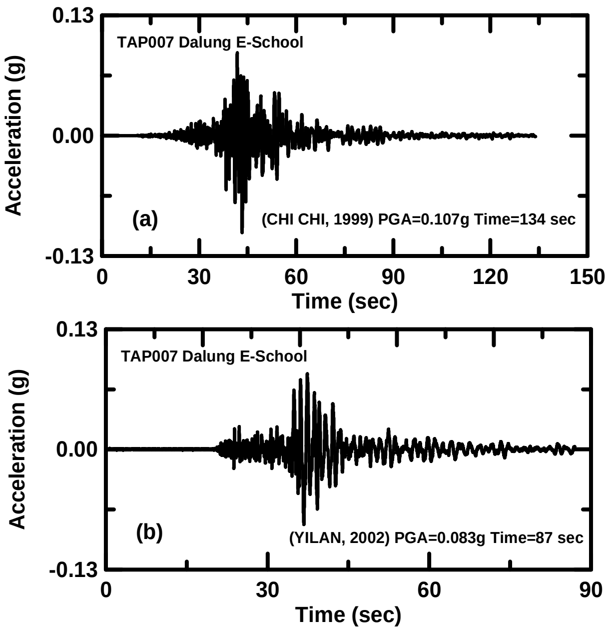

Figure 6. Accelergram records of (a) Chi-Chi and (b) Yi-Lang EQs

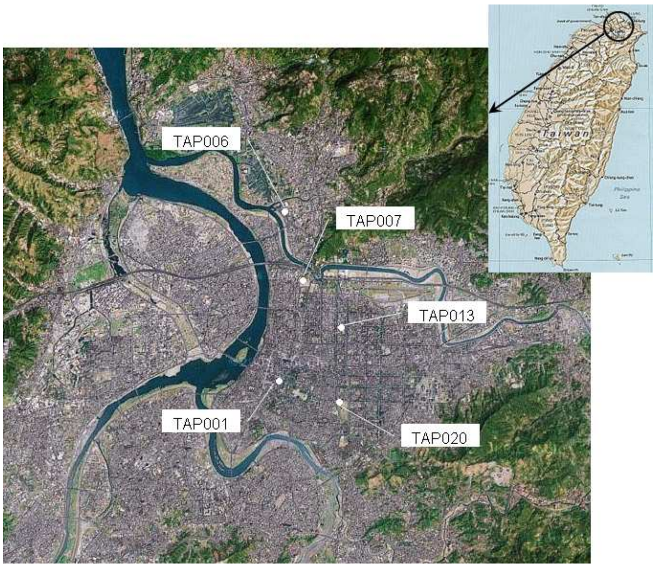

Figure 7. Selected seismic stations in Taipei City (from http://ericyu.org/map/)

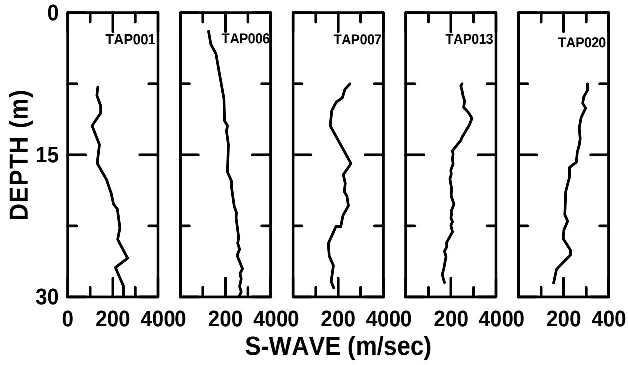

Figure 8. Velocity profiles at the seismic stations at Taipei City

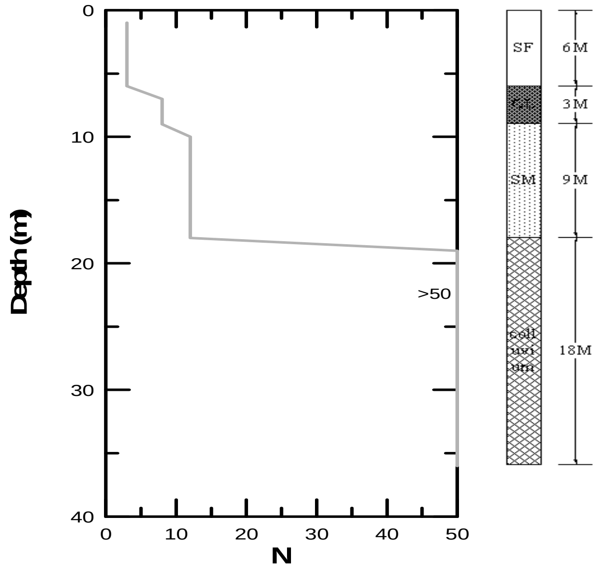

Figure 9. Ground profile used at the site of pile foundation

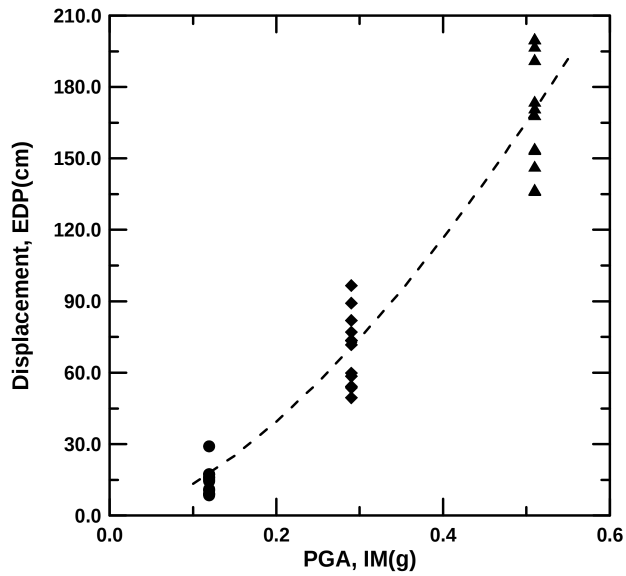

Figure 10. Demand curve of EDP medians based on target PGAs

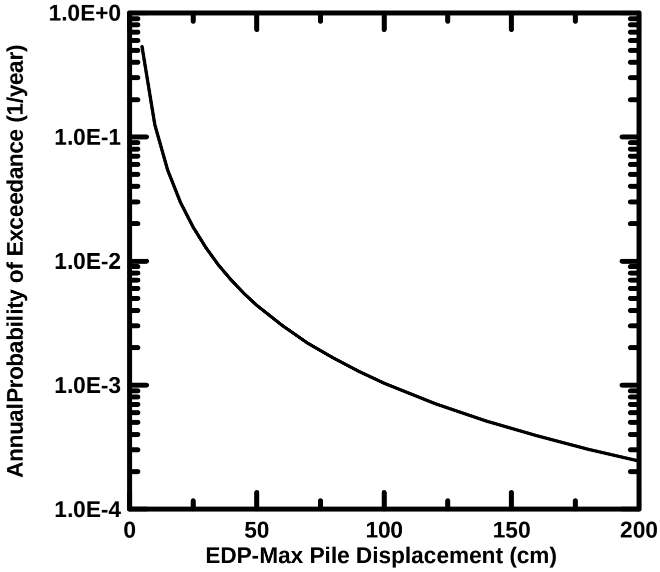

Figure 11. Mean annual rate of exceedance for various EDPs

The uncertainties, β of a certain *edp* could be computed by summing up the uncertainties of these three events. Finally, all the parameters can be substituted into Eq. (4) to compute for λ_{EDP}. Figure 11 presents the annual rate exceedance for various EDPs. EDP of 20, 76 and 168 cm are corresponding to the return periods of 30, 475 and 2500 years. One can simply take these values and compare them to the designated values (if available) for possible PBD assessments. Further comparisons could also be done for damage and loss models. Figure 12 illustrates the results for internal moments obtained by PBEE and EQWEAP analyses. If the critical moments of the pile can be found, then one can easily determine the limits of the pile displacements. Such limits can be regarded as the indices for pile design purpose. These results can help one to conduct the seismic PB analysis for the piles.

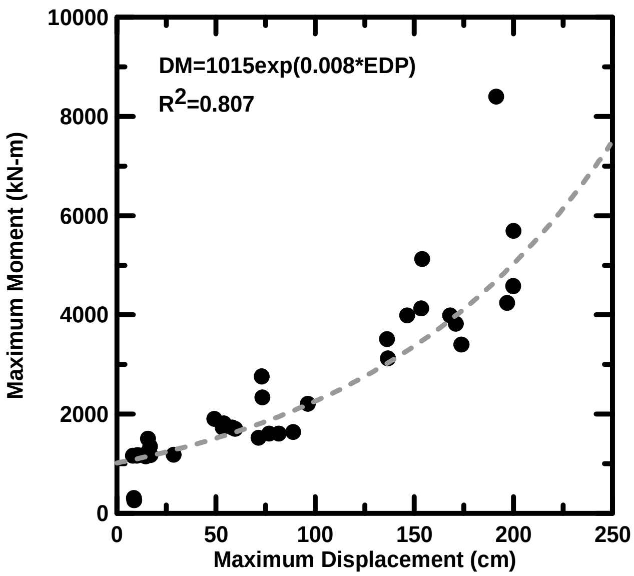

Figure 12. Maximum bending moments vs. maximum pile displacements for single pile located in Taipei Basin

7. CONCLUDING REMARKS

It is shown in this paper that the PBEE analysis suggested by US PEER can be adopted to analyze for the seismic performances of the piles. Incorporating the simplified form of this procedure with the one-dimensional EQWEAP analysis for seismic responses of the piles, a single pile located in Taipei Basin is analyzed considering mainly the horizontal earthquake excitations. The structural parameters such as the maximum pile displacements and the internal bending moments were computed at various seismic design levels. It is pointed out that the design measured should be kept within a certain limits based on the pile performance. One can manage the design by restricting the annual rate of exceedance for the structural parameters in demand and/or by limiting the structural displacements upon the damages. The safety of the piles based on their strength capacity could be analysed too according to the procedures. The details of these design criteria however require more studies and attentions. For incorporation of these analyses onto the whole pile foundation, the superstructural loads and the interactions between the piles and the cap-pile-soil also need to be included in order to obtain more realistic results. The proposed analysis applies only to single piles whereas the ground conditions were known based on available data from the site investigations.

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