Seismic PBD of Piles from Monte Carlo Simulation Using EQWEAP Analysis with Weighted Intensities

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ABSTRACT: This paper discusses the seismic performance based design (PBD) analysis on piles using one-dimensional stress wave equation and Monte Carlo Simulation. Seismic responses of the piles were monitored at a wider spectrum of earthquake intensities rather than the target ones. To obtain appropriate estimations, weights of the intensities were calculated from the probability density function solvable from the seismic hazard curve. Probabilities of failure of the piles were evaluated for uncertainties of soil parameters and seismic records, and then calibrated with the weights. The result of the numerical study indicates that the seismic force is the most dominant factor. Large diameter pile will exert cracks around pile head under moderate earthquakes. Therefore assessment based on only the pile head would become very critical. For design and maximum consideration (MCE) earthquakes, the piles were found satisfied because of performance required on ductility resistance and ultimate moment capacity. Probabilities of failure of the piles were also found sensitive to horizontal load from the superstructure. Comparing the correspondent reliability indexes with those required for acceptable foundations, the seismic performance of the piles can be assessed. With the suggested factor of safety, the seismic performance of the piles was found to be 1.1~2.2 for design and MCE quakes in this study.

1. INTRODUCTION

In the past decade, Performance Based Design (PBD) has received considerable attentions from geotechnical engineering societies. Engineers can follow the guidelines in AASHTO LRFD, Eurocode-7 and -8, and Geocode-21 to conduct the analyses. For design of pile foundations, the uncertainties of design factors and influences on stabilities and deformations of the piles were considered. The effects of loading, soil parameters as well as the spatial variability need to be taken into account. In addition, the influences of calculation methods, tests and measurements, and construction methods deserve further attentions. In recent years, PBD on pile foundation has been discussed using the Reliability method (Paikowski, 2004; Phoon, 2008). In the design practice, seismic PBD of the piles is mostly conducted using static and pseudo static analyses. The pile is generally analyzed with the largest ground motion or with a displacement profile at arbitrary time obtained prior to the analysis. Performance of the pile at the target peak ground acceleration (PGA_t) can be assessed with the design requirement. Although it is relatively simple, such analysis seems to provide very safe and conservative solutions for engineering practice. For more realistic solutions, a dynamic modelling is preferred. In this paper, a dynamic solution based on EQWEAP analysis (Chang *et al.*, 2006, 2008 and 2014) is adopted. Monte Carlo Simulation method is used to analyze the reliability index for design.

2. SEISMIC PERFORMANCE BASED DESIGN OF PILES

For any geotechnical structure, the PBD analysis can include two issues, 1. bearing capacities, and 2. deformations of the structure. Honjo et al. (2002) suggested that the analysis can be done using 1. LRFD Method, 2. Reliability Method, and 3. Probability Method. Applications of the first two methods were frequently presented on pile foundations (Paikowski, 2002 and 2004; Honjo and Nagao, 2007; Phoon, 2008; Zhang, 2008). Performance of the piles at both ordinary and seismic conditions can be evaluated. Usually in the seismic cases, only a few seismic intensities (e.g., PGA) from the design code were considered. The uncertainties are mostly related to seismic records, soil parameters and spatial variability. The applicable reliability methods, e.g., First Order Second Moment (FOSM), First Order Reliability Method (FORM) and Monte Carlo Simulation (MCS) are commonly adopted to estimate the probability of failure and reliability index of the pile. Figure 1 illustrates the possible methods used for seismic PBD of pile foundations.

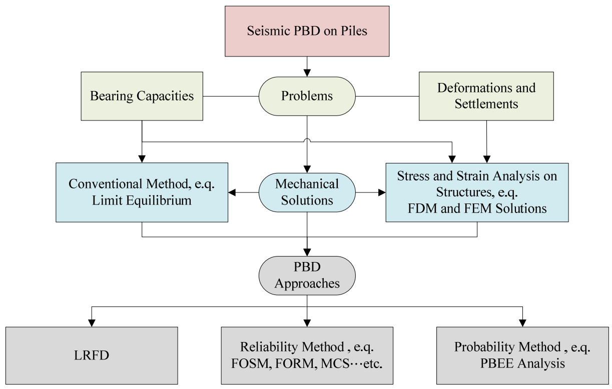

Figure 1 Problems and methods in seismic PBD of pile foundations

2.1 Probability approach based on PBEE analysis

For Probability Based method, the PBEE (Performance Based Earthquake Engineering) analysis suggested by US Pacific Earthquake Engineering Research Centre (PEER) is referable. Such analysis has been conducted by research teams on NEES project with 3D FEM program-OpenSees (2009). Excellent overview of the PBEE analysis can be found in Kramer (2008). It suggests that the annual rate of exceedance (λ) for a decision variable (DV) on any engineering structure can be analysed as a triple-integral on probabilities of intensity measured (IM), engineering demand parameter (EDP), and the damage measure (DM). For seismic hazard curve in hand, the integral is able to decompose to find the individual rate of exceedance for EDP, DM, and DV, respectively. Based on log-normal distributions, analytical expressions of the rate exceedance for EDP, DM and DV can be found at different seismic levels. Simplified methods to compute the statistics of the data were suggested by Kramer (2008). One can easily follow the procedures to assess the seismic PBD for any geotechnical structure. This approach can ideally include all the possible earthquake influences with the uncertainties of soil parameters and spatial variability. According to Shin (2007), the effects of the uncertainties of seismic forces are much larger than those from soil parameters and spatial variability. The record-to-record uncertainty was found to be

90%~95% of the total uncertainty involved in the seismic assessment.

2.2 EQWEAP analysis

Seismic performance of the piles can be monitored using either static (or pseudo-static) or dynamic analysis. While the former is easier to conduct, the latter requires longer computation time and pre-processing. In order to reduce the time for dynamic computations, a rather fast solution EQWEAP was suggested by Chang *et al.* (2006, 2008). It solves the free-field ground responses (with the lumped mass analysis) and using them to obtain the corresponding pile deformations from 1-D wave equations. With such solution, the PBD analysis based on dynamic modelling becomes more applicable. Details of the EQWEAP analysis can be found in Chang et al. (2014). This solution was found agreeable with 2D and 3D FEM analyses using PLAXIS (2012) and Midas-GTS (2012) with simple geometry conditions (Chang *et al.*, 2013a). Figure 2 illustrates the numerical schemes used for EQWEAP analysis.

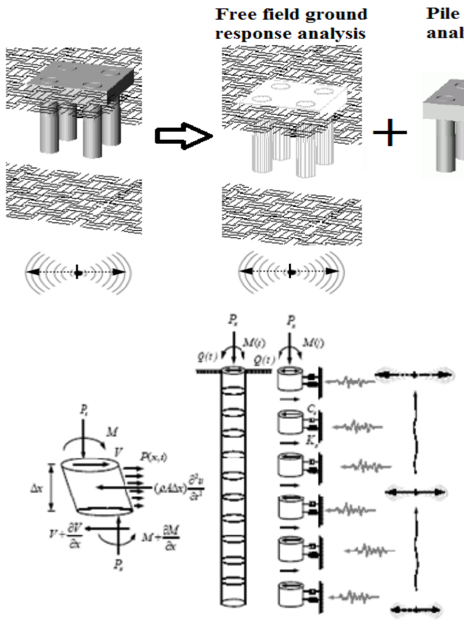

Figure 2 Numerical scheme used in EQWEAP analysis

2.3 Case study from EQWEAP and PBEE analyses

The EQWEAP analysis has been combined with the PBEE procedures (Chang *et al.*, 2009, 2010) in evaluating the seismic performance of piles in Taipei. The dynamic impacts of the ground motions can be monitored through this solution. According to the local design code, seismic level-I, -II and -III required for moderate earthquake, design earthquake and the maximum consideration earthquake (MCE), respectively were considered. For 50-year design life, the corresponding probabilities of the occurrence of these quakes are 80%, 10% and 2%, whereas the seismic return periods are 30, 475 and 2500 years.

Figure 3 reveals the local seismic hazard curves suggested by Cheng (2002), the corresponding target PGA_t at these seismic levels in Taipei were reported as 0.12g, 0.29g and 0.51g.

With the target PGA_t and acceleration records from nearby stations, seismic PBD on bridge piles (D=2m and L=60m) of an expressway at the Sin-Jhuang District in Taipei was analyzed by Chang *et al.* (2013[b, c, d]). Tri-linear moment-curvature relationship of the concrete pile was suggested based on the approximate Bouc-Wen model (Kunnath and Reinhom, 1989) (see Figure 4). To keep the piles remain elastic at moderate earthquakes, the maximum bending moment (M_{max}) needs to be less than M_{cr}, at which the concrete starts to crack. For design EQ, M_{max} should be less than M_y, where the steel bar starts to yield. For MCE quakes, the ultimate moment, M_{ult} in which the plastic hinge occurs must not be exceeded. Note that the curvature of pile deformations is denoted as φ. By analyzing experimental data or results of analytical computations, one can apply the model equation to compute parameters α and Z for these line-segments and then find out the correspondent secant stiffness, EI for the use of nonlinear analysis.

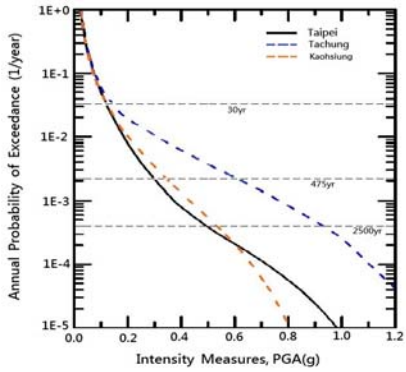

Figure 3 Local seismic hazard curves in Taiwan (after Cheng, 2002)

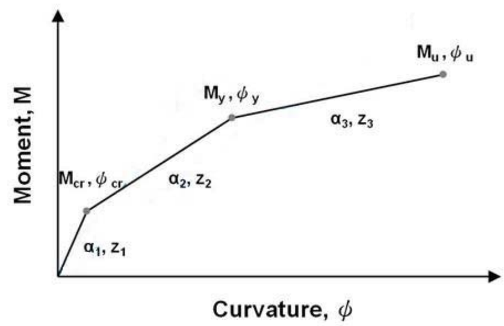

Figure 4 Nonlinear moment-curvature relationships of concrete piles

By taking the internal moments of pile as DM and comparing it with these moment capacities (i.e., M_{cr}, M_y and M_{ult}), the engineers can evaluate the seismic PBD of piles. Figures 5a and 5b reveal the result for seismic assessments of the piles in Taipei Basin (details of the site conditions and pile foundation are described in following paragraphs). In Figure 5b, the maximum absolute pile displacements (solid points in dash curve) found at the pile head are about 20, 50 and 80cm, respectively for moderate, design and MCE quakes. In Figure 5a, comparing the maximum pile moments (solid point in dash line) with the moment capacities shows that the seismic level-I is difficult to satisfy since the predicted maximum bending moment at the pile head will exceed M_{cr} at moderate earthquakes. On the other hand, matching the requirements for design and MCE quakes are relatively easier providing that M_{cr} and M_{ult} of the pile are carefully estimated. The engineers can also use these figures to find the allowable pile displacements under different seismic levels, i.e., U_{mc}, U_{my} and U_{mm}.

3. MONTE CARLO SIMULATION VARYING SEISMIC FORCE

3.1 Weighting intensities from probability densities of the hazard curve

As mentioned before, one can simply conduct the Reliability analysis for a structure at the target PGAs found by the probabilistic seismic hazard analysis (PSHA), the results are generally acceptable in engineering practice. Such approach is often followed by LRFD and other type design methods. By doing so, the seismic influences of the earthquakes are only focusing on a few target intensities rather than all the possible ones. In considering the design life of the structure, it will be more objective to include all quake influences rather than a few ones.

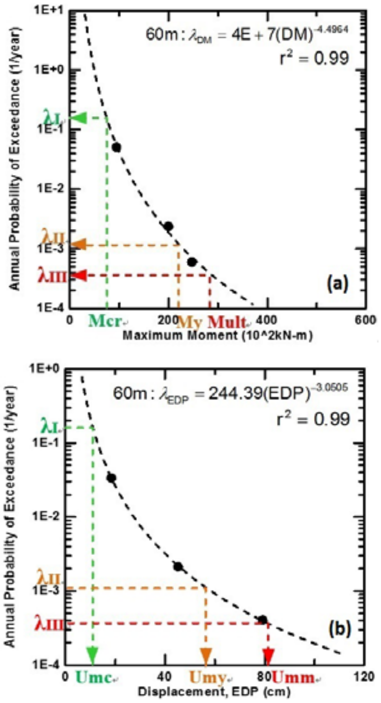

Figure 5 Rate of exceedance for the maximum displacements and bending moments of the piles (after Chang *et al.* 2013[b,c,d])

To do so, the influences of every PGAs can be found from the seismic hazard curve. The annual rate of exceedance, λ of any PGA is subtracted from 1.0 to obtain the probability of occurrence for any intensity less than or equal to that PGA during the design life, i.e., the cumulated density function (CDF). The probability density function (PDF) of the PGAs can be found by differentiating CDF. For the seismic intensities from "not noticeable to person" to "danger to structure", one can thus compute the weights of these intensities and use them to calibrate the corresponding probability of failure for the structure. The probability of failure is strongly dependent on the performance function of the designed seismic level. With the central difference formulas, the weights of every possible PGA can be calculated from the PDF as follows,

$$P_A(a) = \frac{d}{da}F_A(a) = \frac{d}{da}(|1-R_A(a)|) = \frac{dR_A(a)}{da} \quad (1)$$

where $P_A(a)$ is the probability density function, F_A is the cumulated density function, $R_A(a)$ is the function of seismic hazard curve, and a is the variable of intensity measure. Table 1 lists the weights for PGAs from seismic hazard curve for Taipei as shown in Figure 3. Note that the range of PGAs is selected from 0.01g (local intensity of IIV) to 0.51g (PGA_t at MCE, local intensity of VII). Note that the summation of these weights is equal to one.

3.2 Monte Carlo simulation

The accuracy of Monte Carlo Simulation (MCS) depends on the number of simulations considered. For seismic intensities between 0.01g~0.51g, an increment of 0.01g is used. Totally 51 intensities (PGAs) were analyzed for the uncertainties of seismic forces. Alternative methods to produce the seismic records for the structural response analysis have been discussed by Kramer (1996). For simplicity, the acceleration time history recorded at a near-by seismic station can be used to produce the bedrock motions with any target PGA (PGA_t). Varying with other variables (e.g. material parameters and geologic conditions), the database of MCS can be optimized. The total probabilities of failure, P_{fT} at different seismic levels (PGA_t) with the required performance can be computed summing up all the individual probabilities of failure at the PGAs less than and equal to the PGA_t.

$$P_{fT} \text{ at any } PGA_t = \sum P_{fi} \text{ where } a \le PGA_t \quad (2)$$

Assuming that the probability densities of the maximum pile moments at the PGAs can be analyzed as normal or log-normal distributions, the reliability index, β of the MCS can be achieved. Note that β can be calculated by

$$\beta = \mu / \sigma \quad (3)$$

where μ is the mean value and σ is the standard deviation of the data.

4. EXAMPLE STUDY

4.1 Numerical model

Numerical model of a bridge pile foundation at an expressway located in Sin-Jhuang district at New Taipei City is again studied. Based on the previous study by Chang *et al.* (2013[a,b,c,d]), the acceleration records producing largest deformations and smallest deformations of the piles were selected. Stations TAP017 for 1999 Chi-Chi earthquake (in-land/active faulting triggered quake, M_L=7.3) and at TAP011 for 2002 Yi-Lang earthquake (east coast offshore/subduction plate triggered quake, M_L=6.8) were therefore in use. Figure 6 and Figure 7 show their locations and the acceleration time histories used in this study. A typical 3×3 pile foundation with piles of 2m diameter and 60m length was investigated. The *EI* value of the pile is 2.4×104 MPa. Geological condition of the site is presumed based on in-situ borehole data and studies made by Wu (1988) and Hwang *et al.* (2013). Table 2 shows the information of the ground site. According to the designer, the maximum vertical loads at the single piles were designed as 9MN and 18MN for ordinary and seismic cases. Horizontal loads were kept as 15% of the maximum vertical loads. The moment capacities of M_{cr}, M_y and M_{ult} were able to obtain from LPILE analysis (Reese and Van Impe, 2001). With 1.94% reinforced bar ratio and 18MN vertical loads, M_{cr}, M_y and M_{ult} were obtained as 7.35, 22.15 and 28.68 MN-m, respectively. Varying the unit weight, SPT-N value, friction angle and cohesion of the layered soils with presumed averages and standard deviations (see Table 2), 5000 combinations of the soil layers were randomly generated. Varying 50 PGAs on two seismic records, the total number of simulations is 5×10^5. Note that according to the local seismic design code for seismic level-I, -II and -III, the performance functions depend on M_{cr}, M_y and M_{ult} respectively. The EQWEAP analysis is efficient to provide fast solutions within limited time period. The probability of failure, P_f is defined as the ratio of number for cases at failure (defined by $M_{max} \ge M_{cr}$ or $\ge M_y$ or $\ge M_{ult}$) divided by the total number of cases. The reliability indexes, β are computed accordingly.

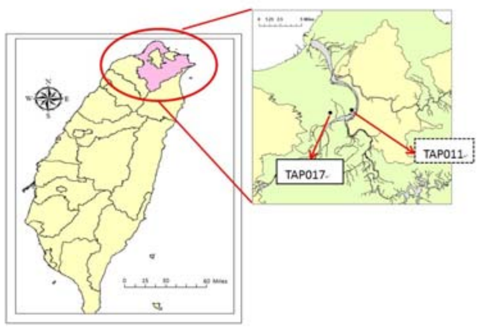

Figure 6 Locations of seismic stations near to the pile foundation site

Table 1 Calculated weights of the intensities for PGAs in between 0.01g~0.51g

PGA (g)	Return period (year)	λ(%)	Probability of occurrence for *a* > PGA	Probability of occurrence for *a*≤ PGA	Numerator of the central difference formula	Weights
0.01	1	100.00	1.0	0.000	5.00E-03	2.50E-03
0.02	1.005	99.50	0.995	0.005	1.00E-02	5.00E-03
0.03	1.01	99.00	0.99	0.010	4.95E-01	2.48E-01
0.04	2	50.00	0.50	0.500	7.50E-01	3.75E-01
0.05	4	25.00	0.250	0.750	3.33E-01	1.67E-01
0.06	6	16.67	0.167	0.833	1.25E-01	6.25E-02
0.07	8	12.50	0.125	0.875	6.67E-02	3.33E-02
0.08	10	10.00	0.100	0.900	5.36E-02	2.68E-02
0.09	14	7.14	0.071	0.929	5.00E-02	2.50E-02
0.10	20	5.00	0.050	0.950	2.98E-02	1.49E-02
0.11	24	4.17	0.042	0.958	1.67E-02	8.33E-03
0.12	30	3.33	0.033	0.967	1.31E-02	6.55E-03
0.13	35	2.86	0.029	0.971	9.52E-03	4.76E-03
0.14	42	2.38	0.024	0.976	8.57E-03	4.29E-03
0.15	50	2.00	0.020	0.980	7.42E-03	3.71E-03
0.16	61	1.60	0.016	0.984	6.11E-03	3.06E-03
0.17	72	1.40	0.014	0.986	5.03E-03	2.51E-03
0.18	88	1.14	0.0114	0.9886	3.89E-03	1.94E-03
0.19	100	1.00	0.0100	0.990	3.36E-03	1.68E-03
0.20	125	0.80	0.0080	0.992	3.01E-03	1.50E-03
0.21	143	0.70	0.0070	0.993	1.90E-03	9.51E-04
0.22	164	0.61	0.0061	0.9939	1.73E-03	8.65E-04
0.23	190	0.53	0.0053	0.9947	1.57E-03	7.86E-04
0.24	221	0.45	0.0045	0.9955	1.29E-03	6.47E-04
0.25	252	0.40	0.0040	0.996	1.03E-03	5.14E-04
0.26	286	0.35	0.0035	0.9965	9.65E-04	4.83E-04
0.27	333	0.30	0.003	0.997	9.33E-04	4.66E-04
0.28	390	0.26	0.0026	0.9974	8.98E-04	4.49E-04
0.29	475	0.21	0.0021	0.9979	5.64E-04	2.82E-04
0.30	500	0.20	0.002	0.998	2.30E-04	1.15E-04
0.31	533	0.19	0.0019	0.9981	2.61E-04	1.30E-04
0.32	575	0.17	0.0017	0.9983	2.50E-04	1.25E-04
0.33	615	0.16	0.0016	0.9984	3.19E-04	1.59E-04
0.34	704	0.14	0.0014	0.9986	3.75E-04	1.87E-04
0.35	800	0.13	0.0013	0.9987	2.80E-04	1.40E-04
0.36	877	0.11	0.0011	0.9989	2.50E-04	1.25E-04
0.37	1000	0.10	0.0010	0.999	2.05E-04	1.02E-04
0.38	1069	0.09	0.0009	0.9991	1.43E-04	7.15E-05
0.39	1167	0.09	0.0009	0.9991	1.35E-04	6.77E-05
0.40	1250	0.08	0.0008	0.9992	1.29E-04	6.43E-05
0.41	1373	0.07	0.0007	0.9993	1.21E-04	6.05E-05
0.42	1473	0.07	0.0007	0.9993	1.17E-04	5.84E-05
0.43	1635	0.06	0.0006	0.9994	1.13E-04	5.66E-05
0.44	1767	0.06	0.0006	0.9994	9.16E-05	4.58E-05
0.45	1923	0.05	0.0005	0.9995	7.35E-05	3.68E-05
0.46	2031	0.05	0.0005	0.9995	5.85E-05	2.92E-05
0.47	2167	0.05	0.0005	0.9995	5.78E-05	2.89E-05
0.48	2301	0.04	0.0004	0.9996	5.39E-05	2.69E-05
0.49	2453	0.04	0.0004	0.9996	3.04E-05	1.52E-05
0.50	2475	0.04	0.0004	0.9996	7.69E-06	3.84E-06
0.51	2500	0.04	0.0004	0.9996	4.79E-05	2.39E-05

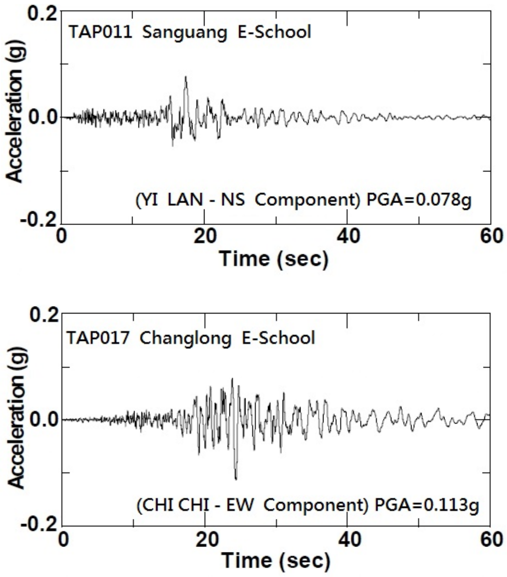

Figure 7 Acceleration time history selected from stations TAP011 and TAP017

4.2 Observations and discussions

Results for the probabilities of failure at each PGA from EQWEAP analysis are shown in Table 3. Note that the static horizontal load from the superstructure is defined as F_H. Calculating the probabilities of failure, P_f at every PGA and multiplying them with the weights, the calibrated P_f can be obtained. Summation of the calibrated P_f up to the target PGA_t gives the total probability of failure, P_{fT} at different seismic levels. It can be obviously seen that the horizontal structural load is very significant to the results. For F_H applied statically, the failures will increase dramatically. If no static horizontal force considered, then the predictions will become much safe. Table 4 reveals the secondary effects of soil parameters varying the number of simulations at each PGA (only the soil parameters were changed). The effects of soil parameters are relatively insignificant compared to PGAs.

The total probabilities of failure are then discussed for different intensity levels following the local scale in Taiwan (CWB, 2000). Table 5 shows the local intensity scale of IV-VII in Taiwan. Table 6 and Table 7 present the total probabilities of failure of the piles under different seismic intensity levels for all possible earthquakes with respect to different PGA_t without and with the static horizontal superstructural load, F_H. It can be found that P_f at the same seismic level will increase with the intensity. Following the assumptions of log-normal distribution, the corresponding reliability indexes, β can be calculated as shown in Table 8 and Table 9. Reliability indexes for seismic performance of the numerical piles were found between 2.8~5 under design earthquake and MCE quakes. Following the foundation performance and reliability index of 2.3 suggested by Whitman (1984), the piles are acceptable for the seismic design requirements. However for moderate earthquake concern, the reliability indexes were found less than 2, which is unacceptable to the design. This is attributed to the fixed-head condition of piles and the static load applied at the pile head. According to Wang (2012), pile head damages would become moderate if the horizontal loads were able to apply with the time dependence.

A factor of safety (FS_P) for seismic performance of the piles from PBEE analysis can be suggested as the ratios of the moment capacities divided by the maximum moments obtained from the PBEE procedures, i.e., M_{cr}/M_{max}, M_y/M_{max} and M_{ult}/M_{max}. Similarly, the factor of safety (FS_R) for seismic performance of the piles from Reliability analysis can be suggested as the ratio of computed reliability index divided by reliability index required β_r, i.e., $\beta_{\text{moderate EQ}}/\beta_r$, $\beta_{\text{design EQ}}/\beta_r$ and $\beta_{\text{MCE}}/\beta_r$. The FS_R and FS_P of the example study are summarized in Table 10. It can be found that the factors of safety calculated from both Reliability and Probability methods showed that the assessment for moderate EQs were over-conservative by only checking the damages at the pile head and taking the horizontal superstructural loads as a static one. The minimum safety factor of the seismic PBD of the piles can be accordingly made at various design levels. In this case, a factor of safety on the order of 1.1~2.2 of the seismic PBD on piles under design and MCE quakes can be suggested.

5. CONCLUDING REMARK

In this paper, Monte Carlo simulation was applied to evaluate the seismic performance of piles from the solutions of EQWEAP analysis. Reliability analysis was conducted varying PGAs at 0.01g~0.51g and soil parameters. The weight of each PGA was able to find by differentiating the cumulated probability density function of PGA. The probability of failure at each PGA was then calibrated with these weights to obtain the total probability of failure for the possible quakes under the target PGAs of the design seismic levels. A numerical model for the pile foundation of an expressway in Taipei was investigated. Two seismic records for acceleration time history were selected based on lessons learned from previous PBEE analyses. For each PGA, 1×104 cases were conducted varying randomly the soil parameters (γ, c, ϕ and SPT-N) of reported data. Uncertainty of the spatial variability was neglected. Following conclusions can be drawn.

1) Seismic PBD of the pile foundation is mainly affected by the seismic forces. The effects of uncertainties on soil parameters were found less important than the seismic forces.
2) For seismic level-I corresponding to the moderate earthquakes, the piles were found very critical due large bending moment encountered at the pile head with fixed head connection. Although the most part of shaft remains in elastic, the occurrence of the cracks at the pile head will fail the assessment. Therefore a compensation of the whole pile behaviours needs to be undertaken.
3) For design earthquake with seismic level-II concern, the piles of the numerical model can sustain the seismic forces and remain under ductility resistance (M_y). For level-III concern under the maximum consideration earthquake, it is found that the internal moments can be controlled under the ultimate moment capacity (M_{ult}).
4) Calibrating the probabilities of failure under the intensities smaller than PGA_t and summing them up, the total probability of failure, P_{fT} at that PGAt can be found. In this study, P_{fT} were reported as 0.054% (96.9%), 0.0 (0.343%) and 0.0 (0.002%) for moderate, design and MCE seismic levels without (or with) the static horizontal superstructural load. The foundation performance for design and MCE concerns were found acceptable based on the suggestion of Whitman (1984). Again, the assessment on seismic level-I will lead to a critical result.
5) The factor of safety (FS) for seismic PBD of the piles was suggested based on the ratios of reliability indexes. It can be found in between 1.1~2.2 for the numerical piles under design and MCE quakes. The factors are slightly greater than those (around 1.1) suggested by PBEE analysis based on ratios of pile moments.

6. ACKNOWLEDGMENT

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Table 2 Geological condition and soil parameters for the numerical model of the site

Depth (m)	H (m)	Soil Layers	γ (kN/m^3)		SPT-N		ϕ (°)		c (kPa)		*Vs* (m/s)
			Avg.	σ	Avg.	σ	Avg.	σ	Avg.	σ	
0~4	4	Surface fill	18.0	1.5	3	1	30	1	--	--	115
4~10	6	SS- IV (ML)	18.5	0.7	5	1	6	1	7	1	171
10~20	10	SS-V (SM)	18.9	1.3	14	3	34	2	--	--	192
20~40	20	SS-IV (CL-ML)	18.8	1	11	2	14	1	5	1	222
40~50	10	SS-III (SM)	18.6	0.9	21	4	34	2	--	--	221
50~60	10	SS-II (CL-ML)	19.0	0.7	14	2	21	1	6	1	241
60~70	10	SS-I (SM)	19.3	0.7	30	4	42	1	--	--	248

NOTE: SS means Songshan formation

Table 3 Original and calibrated probabilities of failure at every intensity measure and their totals

PGA (g)	Moderate EQ/Performance I				Design EQ/Performance II				MCE/Performance III			
	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H
	P_f (%)											
0.01	0.0E+00	0.0E+00	1.0E+00	2.5E-01	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.02	0.0E+00	0.0E+00	1.0E+00	3.8E-01	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.03	0.0E+00	0.0E+00	1.0E+00	1.7E-01	0.0E+00	0.0E+00	1.8E-03	3.0E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.04	0.0E+00	0.0E+00	1.0E+00	6.3E-02	0.0E+00	0.0E+00	2.9E-03	1.8E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.05	0.0E+00	0.0E+00	1.0E+00	3.3E-02	0.0E+00	0.0E+00	2.9E-03	9.7E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.06	0.0E+00	0.0E+00	1.0E+00	2.7E-02	0.0E+00	0.0E+00	2.9E-03	7.8E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.07	0.0E+00	0.0E+00	1.0E+00	2.5E-02	0.0E+00	0.0E+00	2.9E-03	7.3E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.08	5.0E-01	1.3E-02	1.0E+00	1.5E-02	0.0E+00	0.0E+00	3.0E-03	4.5E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.09	6.3E-01	1.6E-02	1.0E+00	8.3E-03	0.0E+00	0.0E+00	4.6E-03	3.8E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.10	7.5E-01	1.1E-02	1.0E+00	6.5E-03	0.0E+00	0.0E+00	1.7E-02	1.1E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.11	8.8E-01	7.3E-03	1.0E+00	4.8E-03	0.0E+00	0.0E+00	3.3E-02	1.6E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.12	1.0E+00	6.5E-03	1.0E+00	4.3E-03	0.0E+00	0.0E+00	3.6E-02	1.5E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.13	1.0E+00	4.8E-03	1.0E+00	3.7E-03	0.0E+00	0.0E+00	3.6E-02	1.3E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.14	1.0E+00	4.3E-03	1.0E+00	3.1E-03	0.0E+00	0.0E+00	3.6E-02	1.1E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.15	1.0E+00	3.7E-03	1.0E+00	2.5E-03	0.0E+00	0.0E+00	3.7E-02	9.4E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.16	1.0E+00	3.1E-03	1.0E+00	1.9E-03	0.0E+00	0.0E+00	5.5E-02	1.1E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.17	1.0E+00	2.5E-03	1.0E+00	1.7E-03	0.0E+00	0.0E+00	1.1E-01	1.8E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.18	1.0E+00	1.9E-03	1.0E+00	1.5E-03	0.0E+00	0.0E+00	1.5E-01	2.2E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.19	1.0E+00	1.7E-03	1.0E+00	9.5E-04	0.0E+00	0.0E+00	1.6E-01	1.5E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.20	1.0E+00	1.5E-03	1.0E+00	8.6E-04	0.0E+00	0.0E+00	1.6E-01	1.4E-04	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.21	1.0E+00	9.5E-04	1.0E+00	7.9E-04	0.0E+00	0.0E+00	1.9E-01	1.5E-04	0.0E+00	0.0E+00	5.0E-04	3.9E-07
0.22	1.0E+00	8.6E-04	1.0E+00	6.5E-04	0.0E+00	0.0E+00	2.3E-01	1.5E-04	0.0E+00	0.0E+00	1.7E-03	1.1E-06
0.23	1.0E+00	7.9E-04	1.0E+00	5.1E-04	0.0E+00	0.0E+00	2.9E-01	1.5E-04	0.0E+00	0.0E+00	2.5E-03	1.3E-06
0.24	1.0E+00	6.5E-04	1.0E+00	4.8E-04	0.0E+00	0.0E+00	3.4E-01	1.6E-04	0.0E+00	0.0E+00	2.8E-03	1.4E-06
0.25	1.0E+00	5.1E-04	1.0E+00	4.7E-04	0.0E+00	0.0E+00	3.6E-01	1.7E-04	0.0E+00	0.0E+00	2.9E-03	1.4E-06
0.26	1.0E+00	4.8E-04	1.0E+00	4.5E-04	0.0E+00	0.0E+00	3.8E-01	1.7E-04	0.0E+00	0.0E+00	2.9E-03	1.3E-06
0.27	1.0E+00	4.7E-04	1.0E+00	2.8E-04	0.0E+00	0.0E+00	4.0E-01	1.1E-04	0.0E+00	0.0E+00	2.9E-03	8.2E-07
0.28	1.0E+00	4.5E-04	1.0E+00	1.2E-04	0.0E+00	0.0E+00	4.4E-01	5.0E-05	0.0E+00	0.0E+00	2.9E-03	3.3E-07
0.29	1.0E+00	2.8E-04	1.0E+00	1.3E-04	0.0E+00	0.0E+00	4.7E-01	6.2E-05	0.0E+00	0.0E+00	2.9E-03	3.8E-07
0.30	1.0E+00	1.2E-04	1.0E+00	1.3E-04	0.0E+00	0.0E+00	5.1E-01	6.4E-05	0.0E+00	0.0E+00	2.9E-03	3.6E-07
0.31	1.0E+00	1.3E-04	1.0E+00	1.6E-04	0.0E+00	0.0E+00	5.4E-01	8.6E-05	0.0E+00	0.0E+00	2.9E-03	4.6E-07
0.32	1.0E+00	1.3E-04	1.0E+00	1.9E-04	0.0E+00	0.0E+00	5.7E-01	1.1E-04	0.0E+00	0.0E+00	2.9E-03	5.4E-07
0.33	1.0E+00	1.6E-04	1.0E+00	1.4E-04	4.8E-01	7.6E-05	5.9E-01	8.3E-05	0.0E+00	0.0E+00	3.0E-03	4.2E-07
0.34	1.0E+00	1.9E-04	1.0E+00	1.3E-04	4.8E-01	9.1E-05	6.0E-01	7.6E-05	0.0E+00	0.0E+00	3.0E-03	3.8E-07
0.35	1.0E+00	1.4E-04	1.0E+00	1.0E-04	4.9E-01	6.8E-05	6.2E-01	6.3E-05	0.0E+00	0.0E+00	3.0E-03	3.1E-07
0.36	1.0E+00	1.2E-04	1.0E+00	7.2E-05	4.9E-01	6.1E-05	6.3E-01	4.5E-05	0.0E+00	0.0E+00	3.1E-03	2.2E-07
0.37	1.0E+00	1.0E-04	1.0E+00	6.8E-05	5.0E-01	5.1E-05	6.5E-01	4.4E-05	0.0E+00	0.0E+00	3.9E-03	2.6E-07
0.38	1.0E+00	7.2E-05	1.0E+00	6.4E-05	5.0E-01	3.6E-05	6.6E-01	4.3E-05	0.0E+00	0.0E+00	5.8E-03	3.7E-07
0.39	1.0E+00	6.8E-05	1.0E+00	6.0E-05	5.0E-01	3.4E-05	6.8E-01	4.1E-05	0.0E+00	0.0E+00	8.9E-03	5.4E-07
0.40	1.0E+00	6.4E-05	1.0E+00	5.8E-05	5.0E-01	3.2E-05	7.1E-01	4.1E-05	0.0E+00	0.0E+00	1.2E-02	6.9E-07
0.41	1.0E+00	6.0E-05	1.0E+00	5.7E-05	5.0E-01	3.0E-05	7.3E-01	4.1E-05	0.0E+00	0.0E+00	1.5E-02	8.7E-07
0.42	1.0E+00	5.8E-05	1.0E+00	4.6E-05	5.0E-01	2.9E-05	7.5E-01	3.4E-05	0.0E+00	0.0E+00	1.9E-02	8.8E-07
0.43	1.0E+00	5.7E-05	1.0E+00	3.7E-05	5.0E-01	2.8E-05	7.7E-01	2.8E-05	0.0E+00	0.0E+00	2.3E-02	8.5E-07
0.44	1.0E+00	4.6E-05	1.0E+00	2.9E-05	5.0E-01	2.3E-05	7.9E-01	2.3E-05	0.0E+00	0.0E+00	2.6E-02	7.6E-07

Table 3 Original and calibrated probabilities of failure at every intensity measure and their totals (continued)

PGA (g)	Moderate EQ/Performance I				Design EQ/Performance II				MCE/Performance III			
	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H	Org. w/o F_H	Cal. w/o F_H	Org. w/F_H	Cal. w/ F_H
	P_f (%)											
0.45	1.0E+00	3.7E-05	1.0E+00	2.9E-05	5.0E-01	1.8E-05	8.1E-01	2.3E-05	0.0E+00	0.0E+00	2.8E-02	8.0E-07
0.46	1.0E+00	2.9E-05	1.0E+00	2.7E-05	5.0E-01	1.5E-05	8.2E-01	2.2E-05	0.0E+00	0.0E+00	3.0E-02	8.0E-07
0.47	1.0E+00	2.9E-05	1.0E+00	1.5E-05	5.0E-01	1.4E-05	8.4E-01	1.3E-05	0.0E+00	0.0E+00	3.2E-02	4.8E-07
0.48	1.0E+00	2.7E-05	1.0E+00	3.8E-06	5.0E-01	1.3E-05	8.5E-01	3.3E-06	0.0E+00	0.0E+00	3.3E-02	1.3E-07
0.49	1.0E+00	1.5E-05	1.0E+00	2.4E-05	5.0E-01	7.6E-06	8.6E-01	2.1E-05	0.0E+00	0.0E+00	3.7E-02	8.8E-07
0.50	1.0E+00	3.8E-06	1.0E+00	2.5E-01	5.0E-01	1.9E-06	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
0.51	1.0E+00	2.4E-05	1.0E+00	3.8E-01	5.0E-01	1.2E-05	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
SUM		5.4E-02		9.7E-01		0.0E+00		3.4E-03		0.0E+00		1.9E-05

Table 4 Reliability indexes obtained at different number of simulations (with F_H)

No of cases	Moderate EQ /Performance I	Design EQ /Performance II	MCE /Performance III
	Reliability index		
1000	<1	2.7695	4.2144
2000	<1	2.6871	4.0956
3000	<1	2.6805	4.0957
4000	<1	2.6817	4.0913
5000	<1	2.6871	4.0924
6000	<1	2.689	4.0942
7000	<1	2.6996	4.1064
8000	<1	2.6821	4.0865
9000	<1	2.6943	4.1049
10000	<1	2.7036	4.1156

Table 5 Local seismic intensity scale in Taiwan (from CWB, 2000)

Intensity	gal (cm/s^2)	g (m/s^2)
IV	25~80	0.02~0.07
V	80~250	0.08~0.24
VI	250~400	0.25~0.39
VII	400~	0.40~

Table 6 Total probabilities of failure for the piles (w/o F_H)

Intensity	PGA≤0.12g	PGA≤0.29g	PGA≤0.51g
	Probabilities of failure		
IV	0.00%	0.00%	0.00%
V	8.07%	0.00%	0.00%
VI	N/A	0.0418%	0.00%
VII	N/A	N/A	0.00%

Table 7 Total probabilities of failure for the piles (w/ F_H)

Intensity	PGA≤0.12g	PGA≤0.29g	PGA≤0.51g
	Probabilities of failure		
IV	88.75%	0.06%	0.00%
V	10.83%	0.21%	0.00015%
VI	N/A	0.14%	0.00098%
VII	N/A	N/A	0.0008%

Table 8 Reliability indexes obtained from total probability of failure (w/o F_H)

Intensity	PGA≤0.12g	PGA≤0.29g	PGA≤0.51g
	Reliability index		
IV	5	5	5
V	1.4	5	5
VI	N/A	3.341	5
VII	N/A	N/A	5

Table 9 Reliability indexes obtained from total probability of failure (w/ F_H)

Intensity	PGA≤0.12g	PGA≤0.29g	PGA≤0.51g
	Reliability index		
IV	1	3.25	5
V	1.24	2.87	4.67
VI	N/A	2.98	4.27
VII	N/A	N/A	4.94

Table 10 Factor of safety for seismic performance of the numerical piles

Method	Factor of safety, FS_P and FS_R		
	Moderate EQ	Design EQ	MCE quakes
PBEE	0.702	1.133	1.113
MCS w/o F_H	0.696 (≤0.12g) and 1.08 @ 0.12g	2.17(≤0.29g) and 2.17 @ 0.29g	2.17 (≤0.51g) and 2.1 @ 0.51g
MCS w F_H	0.43 (≤0.12g) and 1.08 @ 0.12g	1.17 (≤0.29g) and 1.6 @ 0.29g	1.79 (≤0.51g) and 2.08 @ 0.51g

Note: β_R is kept as 2.3 according to the suggestion of Whitman (1984)

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