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Dynamic Analyses for Performance-Based Seismic Design of Geotechnical Structures with Examples in Deep Foundations

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ABSTRACT: Performance-Based Seismic design (PBSD) of the geotechnical engineering structures can be evaluated by a number of methods taking into account the uncertainties of the designed influence factors. Despite the fact that the seismic force is known to be a significant factor, the static and/or pseudo static analyses seem to be commonly adopted in design practice. This paper briefly discusses alternate approaches with the emphasis on dynamic analysis. Examples are given by the assessments of two deep foundations located in Taiwan. It can be found that dynamic analysis is rather important to the seismic design problems since it can monitor the details of time-dependent structural responses incorporating both peak ground acceleration and duration of the earthquake. Other than the 3D finite element analysis, the simplified solution from 1D wave equation analysis can be very effective and convenient for PBSD analysis on deep foundation.

KEYWORDS: Dynamic analysis, Seismic performance, Performance based design, Pile foundation

1. INTRODUCTION

Performance-Based Design (PBD) has been introduced to geotechnical engineering society for nearly two decades (ISO, 1998; Honjo *et al.*, 2002; Fajfar and Kawinkler, 2004; Frank, 2007; Kokusho *et al.*, 2009; PEER, 2010; Bolton, 2012). The significant principle of PBD is that the uncertainties involved in the design must be taken into account. The uncertainties involved in the design of any geotechnical structure can be classified as aleatoric uncertainties and epistemic uncertainties. The former could be introduced statistically by natural changing and/or engineering measurements, whereas the latter could be systematically produced by man-made errors and/or limits of the methods. In geotechnical engineering, the influence factors of the design are mostly focusing on ground conditions (e.g., geometry and geology of the site), physical properties and engineering parameters of the soils, and loads and/or deformations of the structure, etc. The uncertainties of these influence factors must be computed and/or considered in a scientific manner whereas the probability of their occurrence and/or the reliability of their quantities should be analysed and then incorporated into the design. In this way, the design of geotechnical engineering structures could be assured by quality controlled procedures. Conventional measure such as the factor of safety based upon engineering experiences and knowledge is no longer used. Figure 1 depicts the difference of PBD and conventional design on geotechnical engineering structures. The purpose of this paper is to show the useful tool in PBD of piles with the seismic concerns.

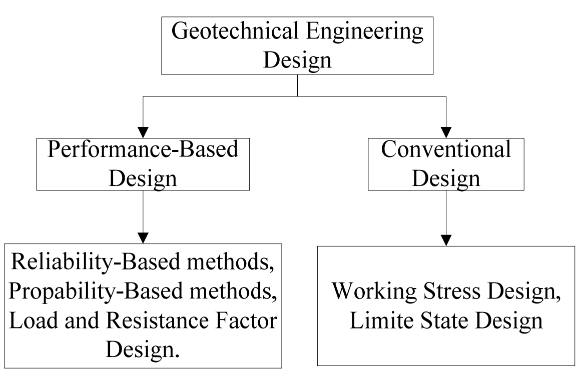

Figure 1 Geotechnical engineering design methods

2. PERFORMANCE OF GEOTECHNICAL STRUCTURES

The performance of geotechnical structures can be analysed on either capacity (and / or resistances) or deformation problems. Different techniques have been adopted to solve the problems. For example, analytical formulas for the capacities of shallow foundation, slope stability, and retaining wall were extensively studied. To take into account the uncertainties, Reliability methods such as the First Order Secondary Moment (FOSM) method, the First Order Reliability Method (FORM), and the Monte Carlo Simulation (MCS) method were adopted in various studies. Corresponding performance function needs to be defined first and the reliability index of the function was calculated accordingly. It was reported that the reliability index should be least 2.4 to satisfy the foundation design (Whitman, 1984). For static capacity performance of the foundation, some people suggested that the physical modelling could also be used. However, it's rarely seen since varying the uncertainties of the influence factors is not easy in the experiment. As to the deformation problems, one needs to conduct the structural analysis and/or the physical modeling. Performance functions in this case can be defined by checking either displacements or stresses (including bending moment) to satisfy the design. The above reliability methods could also be used for the assessment. More information of the applications can be found in Phoon (2008).

Additionally, Honjo *et al.* (2002) suggested that the Probability methods and the Load and Resistance Factor Design (LRFD) method are also available for PBD analysis. Probability methods can be analysed by estimating the probabilities of failure (or occurrence) by a number of consequent measures. Total Probability Theory can be used in such modelling. For LRFD method, load and resistance factors are implemented based upon the AASHTO design specifications. These factors were assumed and evaluated incorporating both the performance function and the reliability analysis to validate the design (Paikowski, 2002). It should be noted that the above methods discussed are mostly suggested to count for aleatoric uncertainties. For epistemic uncertainties, the efforts should be made to gain better knowledge of the system, process of mechanism, in which the methods such as Fuzzy Logic and Evidence Theory are available. Figure 2 summarizes the categories of performance of geotechnical structures and the corresponding analytical procedures on design uncertainties.

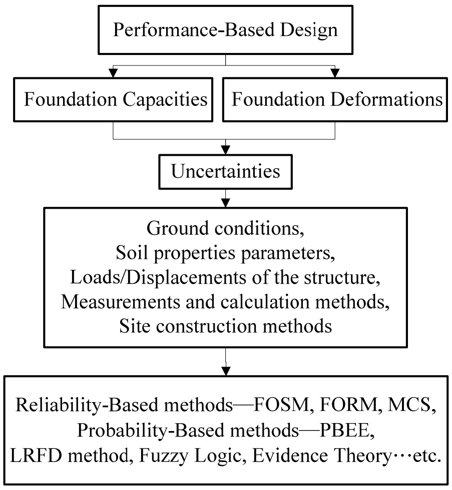

Figure 2 PBD in geotechnical engineering and corresponding analyses

In general, evaluating the structural deformations and stresses is very important to PBD for geotechnical engineering structures. The designer must have a good control of the performance of a foundation. Foundation deformation needs to be controlled to ensure the satisfactions of stresses and bending moments. If deformations of a structure were limited and the structural safety had been ensued, then problems of foundation capacity should become trivial since structural displacements are much less to yield the soils. Recall that the foundation capacities are usually calculated on a hypothetic failure surface occurred in the soils. Although the design guideline of Combined Pile Raft Foundation (CPRF) published by TC212 of ISSMGE in 2013 suggests that both foundation capacities and deformations should be assessed, it should be noted that deformation of a foundation is a key issue rather than the capacity. The capacity problem becomes important only if large displacements of the foundation were encountered. This is especially true in laterally loaded piles and in piled raft foundation encountered large differential settlements. Since the soil model parameters used in calculating foundation capacities will be incorporated in the structural analysis, understanding the material behaviors is thus significant to performance based design.

3. PERFORMANCE BASED SEISMIC DESIGN

For geotechnical structure located in seismic area, the seismic performance needs special attentions. Both numerical modelling and physical modelling are available. The physical modelling including push-over, shake table, and centrifuge tests have been conducted by many researchers. Again varying the influence factors is relatively difficult in physical modelling. On the other hand, numerical modelling is found more economical to the problems. Alternate procedures such as statically push-over simulation, pseudo-static analysis, and the time-dependent dynamic analysis are all applicable. At present time, static and pseudo-static analyses are used more comfortable by engineers. Seismic design of a static ultimate load is known to be conservative. Significance of the dynamic effects upon the structure is generally neglected in these analyses. According to Kramer (2008), the seismic influences of a bridge pile foundation could be dominated by the seismic force rather than any other design factors. With this concern, the uncertainties of the time-dependent seismic force need more attentions. As the computation speed and the capacity of modern computers were improved dramatically, the dynamic analysis becomes a better solution to show the response details.

To count for the uncertainties of the influence factors, the authors have studied the PBEE (Performance Based Earthquake Engineering) analysis (Porter, 2003) and Reliability analysis such as MCS using the dynamic solutions. Example studies can be found in Chang *et al.* (2010 and 2014b). Figure 3 summarizes the alternate solutions associated with the seismic concerns. The influences of the soil parameters and geologic condition are indeed much less than those resulted by seismic forces. It was also found that by calculating the equivalent factor of safety for the seismic design of the pile foundation, the factors of safety obtained from both the PBEE analysis and the MCS method can agree reasonably well. Table 1 shows the definitions of the equivalent factor of safety for seismic design. Their possible values could be in a range of 1.1 to 2.2 for the design and maximum consideration earthquakes.

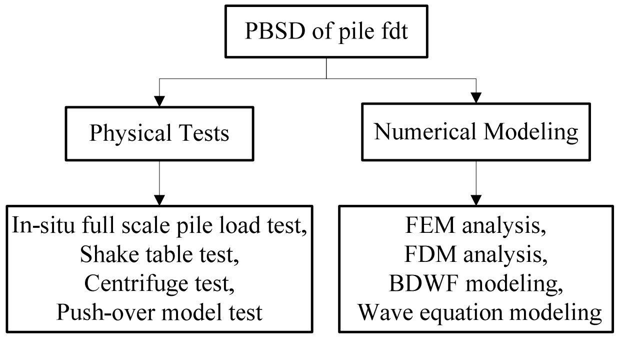

Figure 3 Solutions of PBSD on pile foundation

Table 1 Equivalent factor of safety against seismicity

Method	Factor of safety against seismicity		
	Medium earthquake	Design earthquake	MCE
PBEE analysis	M_{cr} / M_{max}	M_y / M_{max}	M_{ult} / M_{max}
Monte Carlo Simulation	β_{cal} / β_R	β_{cal} / β_R	β_{cal} / β_R

Note: M_{cr} = moment when concrete crack starts; M_y = moment when steel bar yields; M_{ult} = moment when plastic hinge occurs; M_{max} = calculated maximum bending moment; β_{cal} = calculated reliability index; β_R = required reliability index

4. DYNAMIC ANALYSES USING FEM AND EQWEAP

Finite Element Method (FEM) is well known to modern geotechnical engineers. There are a number of FEM packages available to solve the geotechnical problems. To yield rational solutions, discrete mesh and elements as well as the boundary conditions need to be verified. For special interests in the frictions and/or forces between soils and structure, the interface contact elements must be incorporated. In addition, the material models in use are very important. Nonlinear behaviours of the soil and structure can be captured only if appropriate material constitutive laws were used. It was generally found that the stresses obtained from the FE analysis are relatively sensitive in comparison with the displacements computed. At present time, three-dimensional FE analysis is considered as the most rigorous solution for deep foundation behaviours. However it is too time consuming to satisfy the routine design. A recent study carried by Kouroussis *et al.* (2013) has a closer discussion on applying 3D dynamic FE analysis to model the pile-soil-pile interactions.

To simplify the complexity of FE analysis, one-dimensional Finite Difference (FD) formula of the wave equations of single piles under the earthquake excitations has been suggested (Chang *et al.*, 2014a). The corresponding EQWEAP (Earthquake Wave Equation Analysis for Piles) procedure adopts the lumped mass analysis to obtain the free-field ground responses. Once the site responses were obtained, the corresponding pile responses can be computed solving the wave equations of the pile segments. It is necessary to point out although EQWEAP analysis provides one-dimensional time dependent responses of a single pile subjected to dynamic loading, this solution can be extended to two dimensional and / or three dimensional foundation problems providing that the loads from the superstructure were calculated beforehand. If the structural loads were taken into account, the time-dependent dynamic load instead of the static one is recommended to the analysis. A preliminarily discussion of comparing 3D FE and EQWEAP solutions can be found in Lu and Chang (2015).

5. MODELLING ON ARTIFICIAL EARTHQUAKE

For dynamic analysis of a structure under the earthquake excitation, acceleration time history at the underlain bedrock of a ground is required as the input. For convenience, the surface ground accelerations recorded at a nearby seismic station can be used to approximate the bedrock ground motions. The artificial earthquake motions are obtainable with the calibrations upon a peak ground acceleration (PGA) according to the designed requirement. This can be called as the scaling method. The specific PGA values can be determined from the seismic hazard curve that was able to establish through the Probabilistic Seismic Hazard Analysis (PSHA). Solid line and dash lines in Figure 4 illustrate the seismic hazard curves suggested by Cheng (2002) in Taiwan. Nevertheless, local engineers will use more practical values suggested by the seismic design code. The corresponding values following the design code are shown in Table 2.

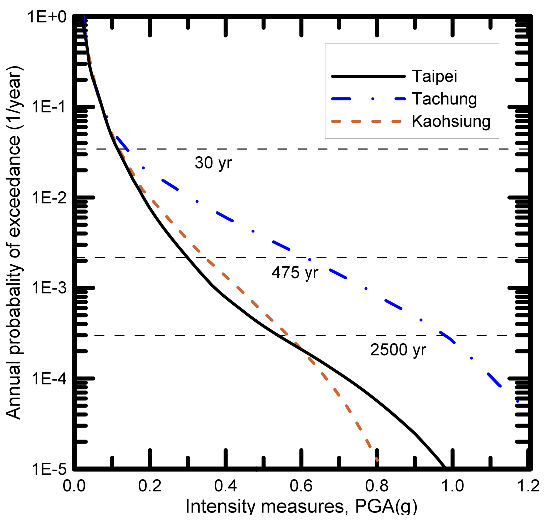

Figure 4 Seismic hazard curves suggested in Taiwan (Cheng, 2002)

Table 2 Target PGA from seismic design code in Taiwan

Metro City	Target PGA (g)		
	Medium Earthquake	Design Earthquake	Max Consideration Earthquake
Taipei	0.06	0.24	0.32
Taichung	0.08	0.35	0.44
Kaohsiung	0.05	0.22	0.28

As mentioned beforehand, the PBEE analysis considers the designed seismic levels at medium, design and maximum consideration earthquakes. Problem arises when selecting the acceleration time history records for the calculations. To overcome such problems, the artificial earthquake record can be obtained by converting a prescribed ground response spectrum to time-domain function. The SIMOKE (Simulator of Artificial Earthquake) analysis suggested by Gasparini and Vanmarcke (1976) is a one to refer. It is then used as the input of the structural analysis. The designed response spectrum of the structure can be simply formed following the seismic design specifications. The converted function is sometimes calibrated with an actual seismic record to yield more realistic solution (Kaul, 1978). One can find more explanations of these methods in Kramer (1996).

Figure 5 indicates some comparisons for the artificial seismic ground motions in Taipei obtained from different techniques. The seismic level of interest is the design earthquake where 0.24g is the target PGA. The difference of the maximum displacements is notable. For any PGA exceeding 0.2g, one needs to be cautious to form the artificial earthquake motion.

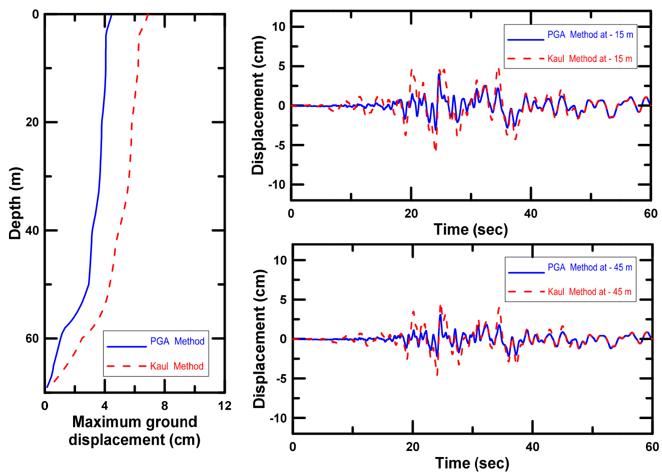

Figure 5 Comparisons of the ground displacements induced by artificial earthquakes

6. CASE STUDIES ON PILE FOUNDATIONS

To give some examples, the deformational seismic performance of the pile foundations were analysed using 3D Midas-GTS program (Midas, 2012) and EQWEAP analysis. The first case study is on a pile-raft foundation of the coal bunkers located at a coal ash pound site near to Taipei. The numerical model approximates the foundation underneath a single coal bunker (see Figure 6). 80 piles of 2 m diameter were oriented in round shape with four radial distances at 7, 14, 21 and 26 meters from the centre pile. The number of piles from the inner ring to the outer ring is 8, 16, 24 and 32. At each edge of the raft, three single piles were seated in a triangle form.

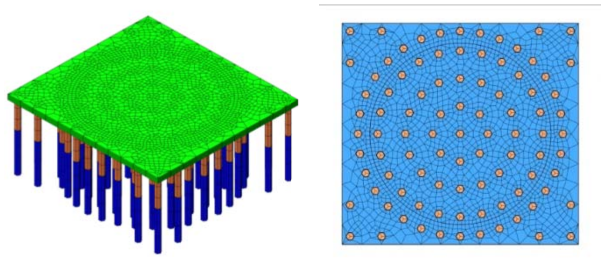

Figure 6 FE layout of pile raft foundation in case study No.1

Table 3 shows the structural dimensions and the material properties / parameters used in the modelling. In the FE analysis, Modified Cam Clay model and Mohr-Coulomb model were used for the coal ash and underlain gravel layers, respectively. The foundation was assumed to be linearly elastic. On the other hand, material nonlinearities were followed in EQWEAP analysis based on program default values. Utilizing the 1999 Chi-Chi earthquake acceleration record at a nearby seismic station (see Figure 7), the displacement-time histories of the P1 pile (centre pile) at the design earthquake were calculated and shown in Figure 8. Note that the superstructure load was temporarily ignored in these analyses. Only the ground motions were considered. Scaling method was adopted to form the artificial earthquake whereas a target PGA of 0.24 g was assigned for the design earthquake. Although the analyses are very different, it was surprisingly to see that compatible solutions were able to obtain. Domination by the ground motions of the structural responses is obviously a reason behind this observation. The PBSD assessment was then conducted using the EQWEAP analysis with PBEE approach.

Table 3 Structural dimensions and material properties/parameters used in the case studies

Case study No.1		
Pile Fdn.	Dimensions	Pile length: 26.5 m; Pile diameter: 2 m ; S/D=2.5⁺; Raft length & width: 60m; Raft thickness: 2 m
	Material properties	E=3x10^4 MPa; ν=0.1; γ= 24 kN/m^3; ξ =0.02
Coal ash and under-lain gravel layer	Geometries	Length and width: 160 m, Coal ash thickness: 12.5 m ; Underlain soil thickness: 16 m
	Coal ash properties/ parameters	Midas analysis: E=20 MPa; ν=0.3; γ=14 kN/m^3; γ_{sat}=17 kN/m^3; cohesion, c=20 kPa; ϕ=35^o; ξ =0.05
		EQWEAP analysis: Same as above; SPT-N=4
	Underlain soil properties/ parameters	Midas analysis: E=2x10^3 MPa; ν= 0.25; γ= 19 kN/m^3; γ_{sat}= 22 kN/m^3; c=0 kPa; ϕ=36^o; ξ =0.05
		EQWEAP analysis: Same as above; SPT-N= 30$^+$
Case study No.2		
Pile Fdn.	Dimensions	Pile length: 28 m; Pile diameter: 0.7 m; S/D= 2.5; Pile cap length & width: 3.9 m ; Pile cap thickness: 0.6 m
	Material properties	E= 3x10^4 MPa; ν=0.1; γ= 24 kN/m^3; ξ =0.02
Sandy gravel	Geometries	Length and width: 100 m, Thickness: 50 m
	properties/ parameters	Midas analysis: E= 97 MPa; ν=0.3; γ= 19 kN/m^3; γ_{sat}= 21 kN/m^3; c=0 kPa; ϕ=38^o, ξ =0.05
		EQWEAP analysis: Same as above; SPT-N= 30$^+$

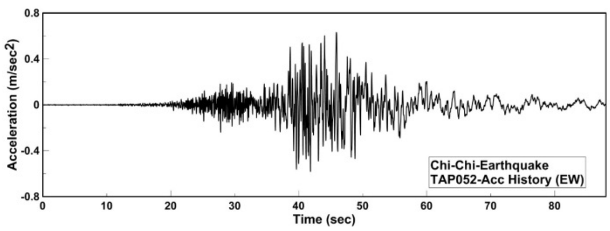

Figure 7 Acceleration time-history used in case study No.1

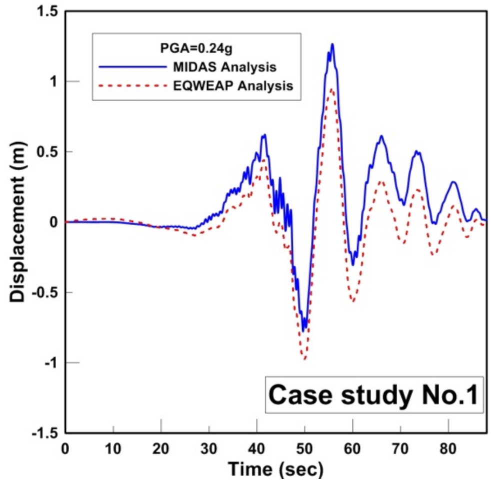

Figure 8 Displacement time history obtained from different analyses for P1 pile in case study No.1

Figure 9 depicts a possible seismic assessment for the maximum pile displacements versus the annual probability of exceedance based on a single seismic record. Typically, at least ten records should be considered if they are available around the engineering site. The absolute values of the maximum pile displacements at different levels of the seismicity in design were found as 0.24, 0.98 and 1.21m in this case. The analyst needs to change the seismic record and use the average and/or the medium values to interpret the PBEE analysis. More detailed discussions can be found in Chang *et al*. (2010).

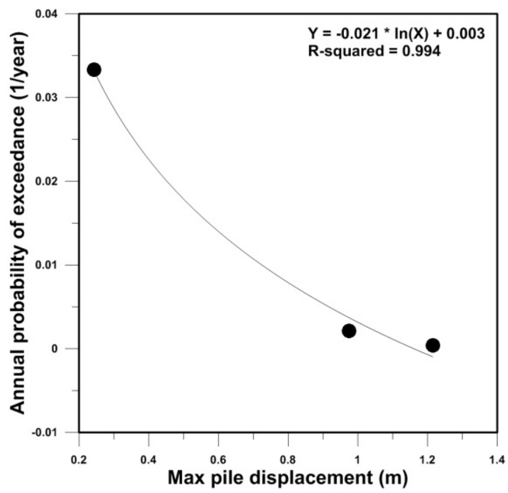

Figure 9 PBEE analysis on maximum pile displacement from EQWEAP for case study No.1

The second case study is on the pile foundations of a 70 meter height statue located at the Da-An coast park in Taichung. Twenty groups of 2×2 pile foundations were designed and oriented in a double-ring shape (see Figure 10). The structural dimensions and the material properties/parameters used in the modelling are summarized in Table 3. For the FE analysis, Mohr Coulomb model was used for the site soils formed by interlayered sand and gravel, the pile is assumed to be linearly elastic. For EQWEAP analysis, material nonlinearities were again monitored. The East-West acceleration time-history record in 1999 Chi-Chi earthquake at a nearby seismic station was used (see Figure 11). Scaling method was adopted for the artificial earthquake whereas the target PGA is aimed at 0.35 g. Neglecting the loads from the superstructure, displacement-time histories of the P2 pile (see Figure 10) were obtained and shown in Figure 12.

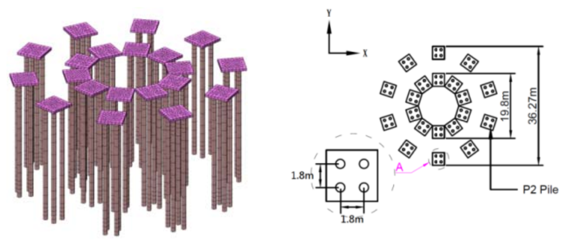

Figure 10 FE layout and bird view of the pile foundations in case study No.2

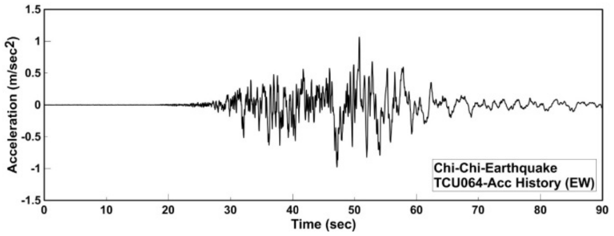

Figure 11 Acceleration time history used in case study No.2

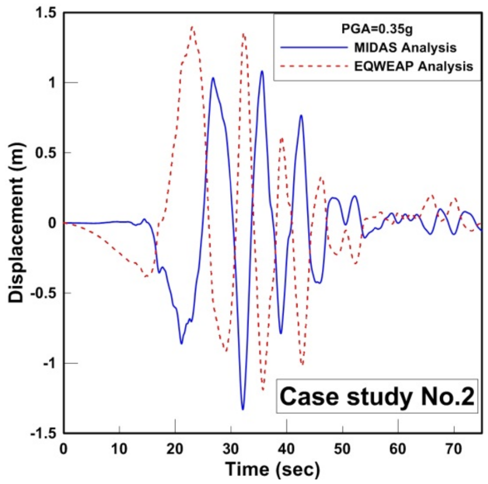

Figure 12 Displacement time history obtained from different analyses for P2 pile in case study No.2

Again, the solutions obtained from different analyses are in similar order. However the response obtained from EQWEAP analysis was found larger than the one from the Midas analysis. It is believed that the material model in use and their parameters are causing such difference. The ones used in EQWEAP analysis need to be calibrations since this site consisting more stiff soils which are somehow inconsistent top the presumptions of the program. Figure 13 shows further seismic assessment following the PBEE procedures using EQWEAP analysis. Similarly, only one seismic record was considered. Note that the absolute values of the maximum pile displacements at different levels of the earthquake are 0.32, 1.4 and 1.75 m in this case. It is necessary to point out that the target PGA is a significant influence factor. The engineers can adjust the pile diameter until satisfaction by checking the flexural deflections and internal stresses of the pile shaft. This procedure should be taken in an efficient manner. To show the applicability of the FE and wave equation analyses, the computation time of both dynamic analyses in two case studies at the design earthquake are shown in Table 4. Time increments of 0.02 sec and 0.002 sec were used for Midas and EQWEAP respectively. It is easy to see that the simplified analysis has great efficiency in the seismic design practice.

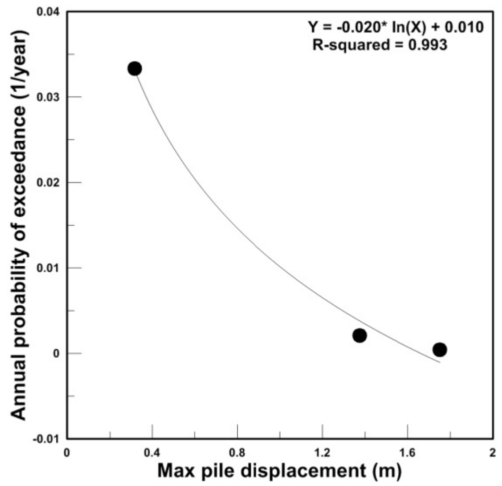

Figure 13 PBEE analysis on maximum pile displacement from EQWEAP for case study No.2

Table 4 Computational time of different dynamic analyses

Case Study \ Analysis	Computational time	
	Midas-GTS	EQWEAP
No. 1	1hr40min34sec for 36153 elements	60 sec
No. 2	7hr14min24sec for 131860 elements	35 sec
Computer Specifications	CPU: Intel Xeon E3-1231v3 RAM: 16GB	

7. CONCLUSION

This paper discusses the alternate methods used to evaluate the performance of geotechnical structures. Examples on evaluating the seismic performance of the deep foundations in Taiwan are given using both the 3D Finite Element analysis and the Finite Difference calculation based on 1D wave equation formulas. Despite the fact that 3D dynamic FE analysis is more rigorous than 1D simplified one, it can be found that the pile displacement time histories obtained from both dynamic analyses have rational agreement. The material model and model parameters are important factors. However, dynamic load induced by the earthquake is more significant in affecting the solutions. The example studies also indicate that the simplified 1D wave equation analysis can be used effectively in design practice. The simplified solution will fasten up the speed of computations, which allows the assessment of PBSD of the structure to be accomplished within a short time. With good measurements of material model parameters and careful preparations of the seismic excitations, the dynamic analysis capable of preserving the time dependence of ground motions and structural responses should be adopted more frequently in PBSD of the geotechnical engineering structures.

8. ACKNOWLEDGEMENT

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