Assessment of seismic performance of soil-structure systems

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ABSTRACT

Three different approaches for assessment of seismic performance of pile foundations (and soil-structure systems in general) are discussed in this paper. These approaches use different models, analysis procedures and are of vastly different complexity. All three methods are consistent with the performance-based design philosophy according to which the seismic performance is assessed using deformational criteria and associated damage levels. It is shown that even though the methods nominally have the same objective, they focus on different aspects in the assessment and provide alternative performance measures. Key features of the three approaches and their unique contribution in the assessment of seismic performance of soil-structure systems are demonstrated using a case study.

1 INTRODUCTION

Methods for assessment of the seismic performance of earth structures and soil-structure systems have evolved significantly in the past two decades. This evolution has involved further improvement of simplified design-oriented approaches, and also development of more robust (and complex) analysis procedures and performance-based design concepts that specifically require evaluation of deformations, permanent displacements and associated damage. A key requirement in the seismic analysis is to achieve this goal while taking into account the uncertainties associated with earthquakes (ground motion characteristics) and complex dynamic behaviour of soil-structure systems.

There are various approaches available for seismic analysis of soil-structure systems ranging from relatively simple practical methods to complex numerical procedures for dynamic analysis. This paper examines three of these approaches as follows:

1) Pseudo-static analysis, a practical approach suitable for conventional assessment and design
2) Seismic effective stress analysis, an advanced method for dynamic time-history analysis of soil-structure systems, and
3) Probabilistic approach for assessment of seismic performance within the so-called Performance-Based Earthquake Engineering (PBEE) framework

The three examined approaches differ significantly in the models they use, required geotechnical data and overall complexity. Importantly, they focus on different aspects in the seismic assessment and treat differently uncertainties and unknowns in the analysis. These methods can be generally applied to various earth structures and soil-structure systems, but here they are applied to the assessment of seismic performance of pile foundations in liquefying soils. The paper illustrates key features of the three approaches and their specific contribution in the assessment of seismic performance of soil-structure systems.

2 OVERVIEW OF BEHAVIOUR OF PILES IN LIQUEFYING SOILS

2.1 Cyclic phase

Soil-pile interaction in liquefying soils involves significant changes in soil stiffness, soil strength and interaction loads over a relatively short period of time during and immediately after the intense ground shaking caused by an earthquake. As illustrated schematically in Figure 1, during strong ground shaking in loose saturated sandy deposits, the excess pore water pressure rapidly builds up until it eventually reaches the initial effective overburden stress, σ_v', and the soil liquefies. In the example shown in Figure 1a (simulated excess pore water pressure at a vertical array site, Kobe, Japan; Ishihara and Cubrinovski, 2005), the excess pore water pressure reached the maximum level after only 6-7 seconds of intense shaking, and this was practically the time over which the soil stiffness and strength reduced from their initial values to nearly zero. The intense reduction in stiffness and strength of the soil was accompanied by an equally rapid increase in the ground deformation, as illustrated with the solid line in Figure 1b where horizontal ground displacement of the liquefied layer is shown. The magnitude of the peak cyclic displacement was about 40 cm which corresponds to an average shear strain in the liquefied layer of about γ_{cyc} = 2.5%. The peak ground displacement occurred nearly at the time when the soil layer liquefied and was accompanied by high ground accelerations of about 0.4g. During this phase of intense ground shaking and development of liquefaction, piles were subjected to kinematic loads due to ground movement and inertial loads caused by vibration of the superstructure, as indicated in Figure 1c. Both these loads are oscillatory in nature with magnitudes and spatial distribution dependent on various factors including ground motion characteristics, soil density, presence of non-liquefiable soil layer at the ground surface, and predominant periods of the ground and superstructure.

2.2 Lateral spreading phase

In sloping ground or backfills behind waterfront structures, liquefaction may result in unilateral ground displacement due to spreading of liquefied soils, as indicated with the dashed line in Figure 1b. Lateral spreads typically result in large permanent displacements of up to several meters in the down-slope direction or towards waterways. Provided that driving static shear stresses exist in the ground, lateral spreading may be initiated either during the intense pore pressure build up, at the onset of liquefaction or after the complete development of liquefaction (during the redistribution and dissipation of excess pore water pressures). Spreading displacements can be one order of magnitude bigger than cyclic displacements while inertial effects are relatively small during lateral spreading (Figure 1c).

3 APPROACHES FOR SEISMIC ASSESSMENT OF PILE FOUNDATIONS

There are various approaches available for seismic geotechnical analysis ranging from relatively simple practical methods for preliminary assessment and design to quite sophisticated numerical procedures for dynamic analysis of earth structures and soil-structure systems. From a seismic-assessment viewpoint, the various types of seismic analysis of piles (and geotechnical analyses in general), can be broadly categorized as follows:

(1) Pseudo-static analysis: this is a practical engineering approach for assessment of piles based on routine computations and use of relatively simple models. This approach aims at estimating the peak value of the dynamic response of the pile under the assumption that dynamic loads can be idealized as static actions. The pseudo-static analysis can be applied to practice without requiring excessive computational resources and specialist knowledge, and hence is a widely adopted approach in current practice and seismic design codes.

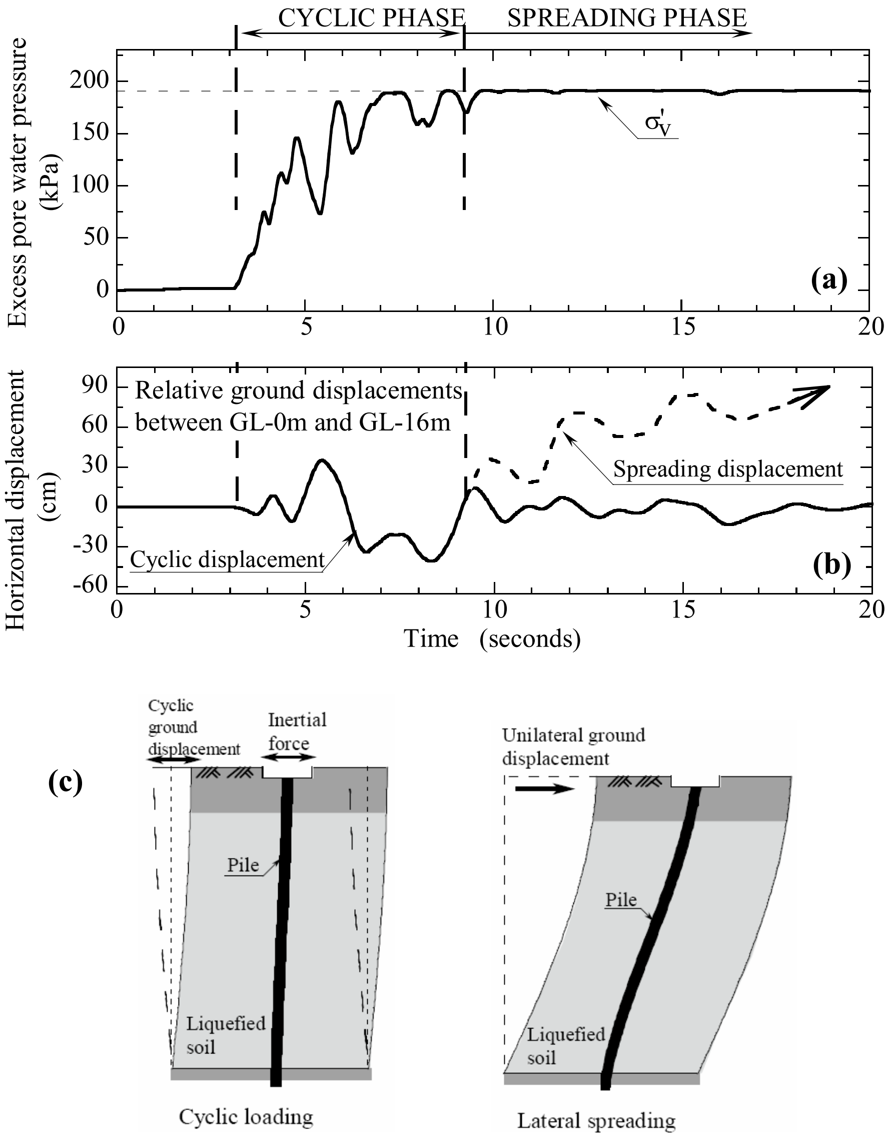


Figure 1: Schematic illustration of ground response and soil-pile interaction in liquefying soils: (a) excess pore water pressure; (b) lateral ground displacement; (c) loads on pile during the cyclic phase and lateral spreading phase

(2) Seismic effective stress analysis: this is a sophisticated method for assessment of the seismic response of soil-structure systems using advanced numerical procedures. The effective stress analysis aims at a very detailed modelling of the complex soil-structure interaction in liquefying soils in a rigorous dynamic analysis. It essentially involves a realistic simulation of the entire process of pore pressure development, onset of liquefaction and post-liquefaction behaviour including associated ground deformation and earthquake loads. The seismic effective stress analysis is generally difficult to apply to practice because it requires significant computational resources and specialist skills from the user. Hence, in concept, it may be considered as the opposite approach to that of the practical pseudo-static analysis.

(3) Probabilistic assessment within the PBEE framework: this is a probabilistic approach for seismic assessment of structures within a so-called Performance-Based Earthquake Engineering (PBEE) framework. In principle, both pseudo-static and dynamic analysis methods can be used as a basic analytical tool employed within the probabilistic assessment. Whereas this approach

undoubtedly introduces additional complexity, it provides a very robust treatment of uncertainties associated with the phenomena considered and analysis procedures. In view of the growing role of probabilistic PBEE assessment in earthquake engineering and its implications for geotechnical engineering practice, it will be considered herein as an alternative approach to the conventional pseudo-static analysis and advanced seismic effective stress analysis.

The above three analysis methods differ significantly in the models they use, required geotechnical data, treatment of earthquake loads and overall complexity. They also impose very different demands in terms of cost, computational time, and required knowledge and skills of the user. These attributes together with the importance of the structure considered are commonly employed as criteria in the selection of the appropriate method of analysis. What is less appreciated is that these analysis approaches provide different viewpoints in the assessment and address uncertainties and unknowns in the analysis differently. In order to illustrate key features and differences amongst the three approaches, these methods will be applied to a case study in the following.

4 CASE STUDY

The Fitzgerald Avenue Bridge over the Avon River in Christchurch, New Zealand, will be used as a case study. Since the bridge has been identified as an important lifeline for post-disaster emergency services, a structural retrofit has been considered in order to avoid failure or loss of function of the bridge in the event of a strong earthquake. In conjunction with the bridge widening, the retrofit involves strengthening of the foundation with new large diameter piles. A cross section of the bridge through the central pier is shown in Figure 2a.

This is a twin bridge with highly variable ground conditions across the foundation soil. According to conventional liquefaction evaluation procedures based on penetration resistance, the thickness of the liquefiable soil varies between 0m and 15m depending on the particular location at the site. Clearly, a rigorous investigation of the seismic response of the bridge and its

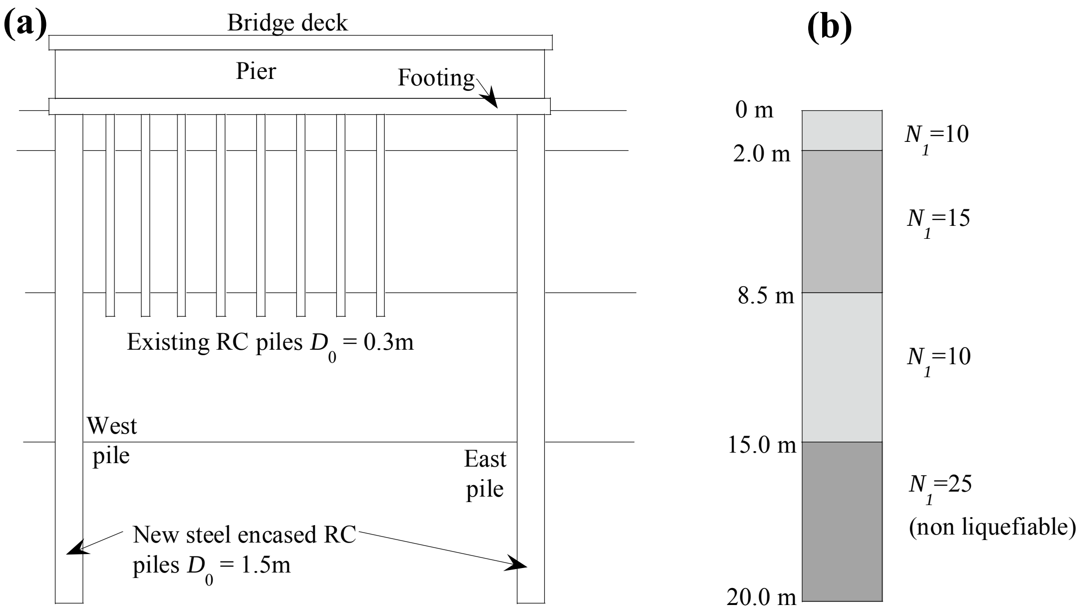


Figure 2: Central pier of the bridge: (a) cross section; (b) soil profile used in seismic effective stress analysis

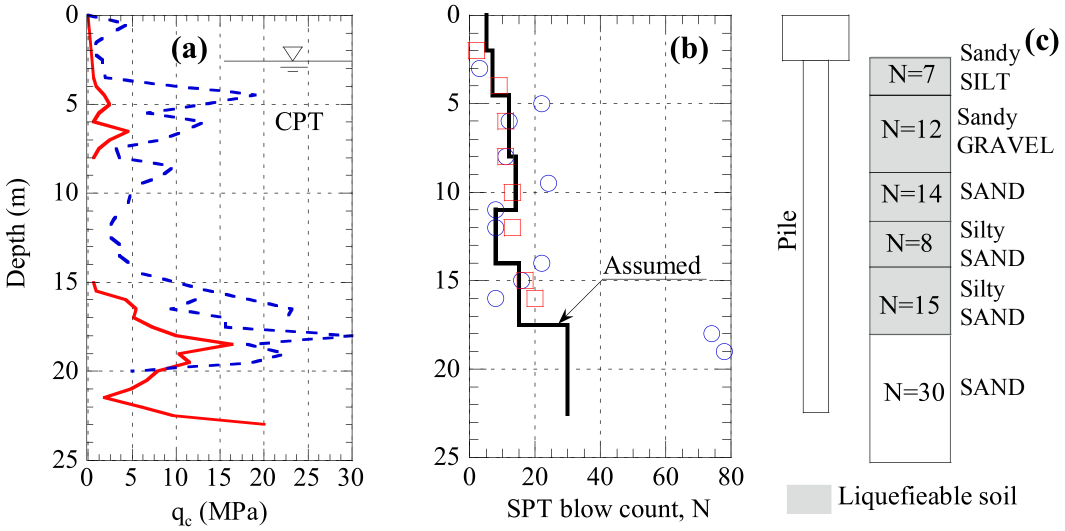


Figure 3: CPT, SPT and soil profile at the north-east abutment of Fitzgerald Bridge

foundation will require consideration of 3-D effects and spatial variability of soil conditions. These complexities are beyond the scope of this paper, however, and rather a simplified scenario will be considered with the principal objective being to examine the response of the pile foundation shown in Figure 2 using the three assessment procedures outlined in the previous section.

Figure 3 summarizes the results of penetration tests (CPT and SPT) at the northeast corner of the bridge including soil stratification. This soil profile was adopted in the analyses. The soil deposit consists of relatively loose liquefiable sandy soils with a thickness of about 15 m overlying a denser sand layer. The sand layers have relatively low fines content predominantly in the range between 3% and 15% by weight. In terms of the normalized SPT blow count, the soil profile was further approximated with four layers, as indicated in Figure 2b.

5 PSEUDO-STATIC ANALYSIS

As a practical approach, the pseudo-static analysis should be relatively simple, based on conventional geotechnical parameters and engineering concepts, and applicable without requiring significant computational resources. The pseudo-static analysis of piles in liquefying soils must also satisfy the following requirements:

- It should capture the essential features of pile behaviour in liquefying soils
- The analysis should permit estimating the inelastic response and damage to piles, and
- It should address the uncertainties associated with seismic response of piles in liquefying soils.

Not all available methods for simplified analysis satisfy these requirements, and most notably, uncertainties in the analysis are commonly either ignored or poorly addressed. In what follows, a recently developed method for pseudo-static analysis of piles in liquefying soils (Cubrinovski and Ishihara, 2004; Cubrinovski et al., 2008) is applied to evaluate the cyclic response of Fitzgerald Bridge pile foundation in order to illustrate important features of the analysis.

5.1 Computational model and input parameters

Although in principle the pseudo-static analysis could be applied to a pile group, typically it is applied to a single-pile model. This is consistent with the overall philosophy for gross simplification adopted in this approach. A typical beam-spring model representing the soil-pile system in the simplified pseudo-static analysis is shown in Figure 4. The model can easily incorporate a stratified soil profile with different thickness of the liquefied layer and a crust of non-liquefiable soil at the ground surface. In the model, the soil is represented by bilinear springs in which effects of nonlinear behaviour and liquefaction are accounted for through the degradation of stiffness and strength of the soil. The pile is modelled with a series of beam elements, which can incorporate a general nonlinear moment-curvature relationship. Parameters of the model are illustrated in Figure 5 where a typical three-layer configuration is shown with a liquefied layer sandwiched between a surface layer and a base layer of non-liquefiable soils. All model parameters are based on conventional geotechnical data (e.g. SPT blow count) and concepts (e.g. subgrade reaction coefficient, Rankine passive pressure). Two equivalent static loads can be applied to the pile in this model: a lateral force at the pile-head representing the inertial load due to vibration of the superstructure, and a lateral ground displacement applied at the free end of the soil springs (Figure 4c) representing the kinematic load imposed by the ground movement.

5.2 Key uncertainties

As described in Section 2, soil-pile interaction in liquefying soils is extremely complex and involves significant and rapid changes in soil stiffness, strength and lateral loads on pile. The key issue in the pseudo-static analysis is therefore how to select an appropriate set of equivalent static values for the soil stiffness, strength and lateral loads, or in other words, what are the appropriate values for β_2, $p_{2\text{-}max}$, U_{G2} and F in the model shown in Figure 5.

Stiffness and strength of liquefied soils

In the adopted model, effects of liquefaction on stiffness of liquefied soil are taken into account through the degradation parameter β_2. Observations from full-size experiments and back-calculations from well-documented case histories indicate however, that for cyclic liquefaction, β_2 typically takes values over a wide range between 1/10 and 1/50 (Cubrinovski et al., 2006).

Similar uncertainty exists regarding the ultimate pressure from the liquefied soil on the pile or the value of $p_{2\text{-}max}$ in the model. The ultimate lateral pressure $p_{2\text{-}max}$ is often approximated using

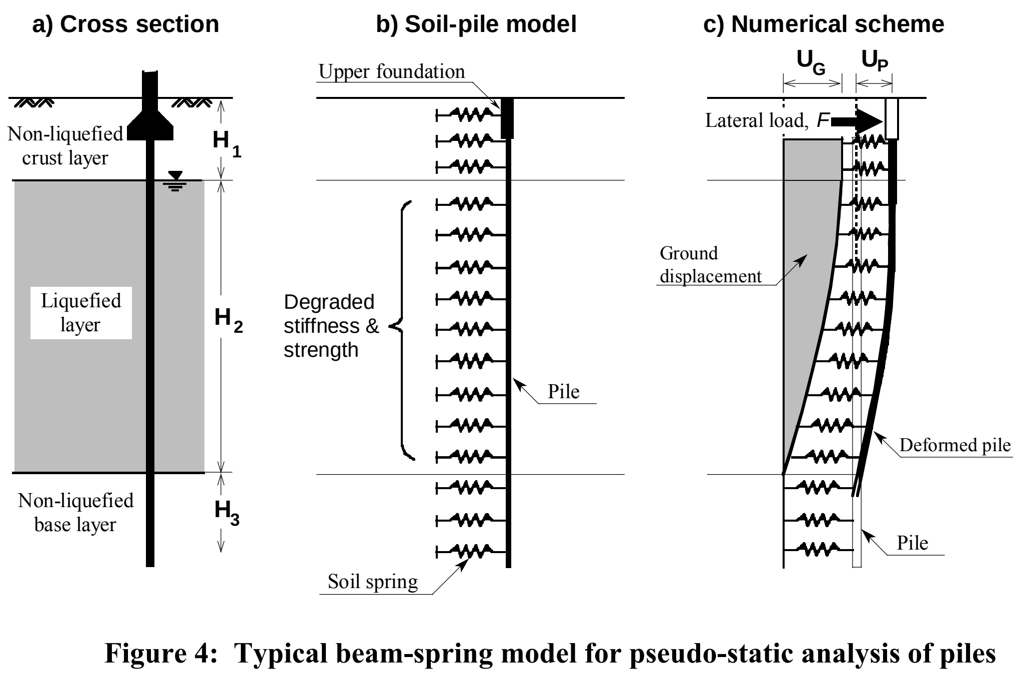


Figure 4: Typical beam-spring model for pseudo-static analysis of piles

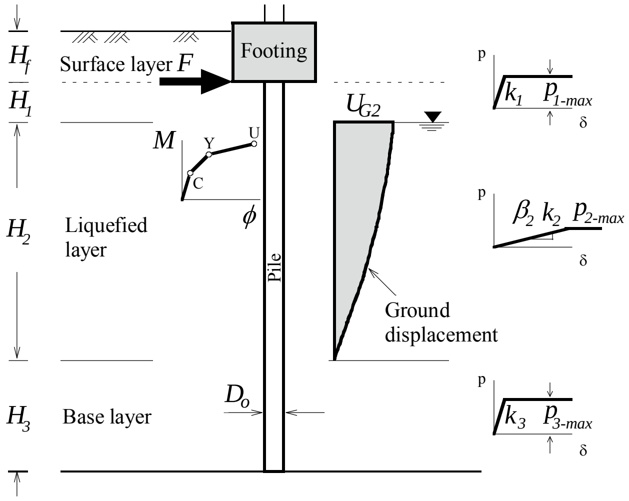


Figure 5: Characterization of nonlinear behaviour and input parameters of the model

the residual strength of liquefied soil (S_r), e.g. $p_{2-max} = \alpha_2 S_r$. There are significant uncertainties regarding both α_2 and S_r values. The latter is illustrated by the scatter of the data indicated in Figure 6 where the most commonly used empirical correlation between the residual strength of liquefied soils and normalized SPT blow count $(N_1)_{60cs}$ (Seed and Harder, 1991) is shown. For a normalized equivalent-sand blow count of $(N_1)_{60cs}$ = 10, for example, the residual strength varies approximately between 5 kPa and 25 kPa.

Lateral loads on pile

The selection of appropriate equivalent static loads in the pseudo-static analysis is the most difficult task in this analysis. The magnitude of lateral ground displacement U_{G2} can be estimated using simple empirical procedures based on SPT/CPT charts (Tokimatsu and Asaka, 1998). Using this method, a value of U_{G2} = 0.36 m was estimated for the Fitzgerald Bridge site. However, since U_{G2} represents in effect an estimate for the peak free field response of the site, it is reasonable to expect a considerable variation in the value of U_{G2} around this estimate.

As mentioned earlier, the objective of the pseudo-static analysis is to estimate the peak value of the dynamic response of the pile. The peak loads on the pile due to ground movement and vibration of the superstructure do not necessarily occur at the same time, and therefore, there is

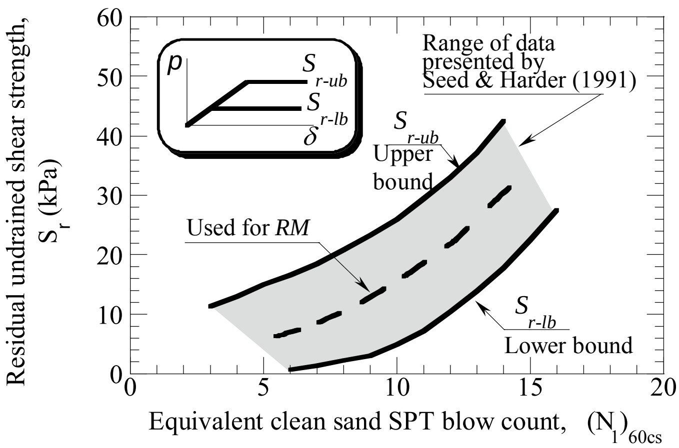


Figure 6: Undrained residual shear strength of sands (after Seed and Harder, 1991)

no clear and simple strategy how to combine these loads. Recently, Boulanger et al. (2007) suggested that the maximum ground displacement should be combined with an inertial load from the vibration of the superstructure proportional to the peak ground acceleration a_{max} using the following expression: $F = I_c m_s a_{max}$. Here, m_s is the mass of the superstructure, Ic is a factor that depends on the period of the earthquake motion, and practically it provides the load combination for U_{G2} and F. Again, a wide range of values have been suggested of I_c = 0.4, 0.6 and 0.8 for a short, medium and long period ground motions respectively.

5.3 Results

A pseudo-static analysis of the new piles (1.5m in diameter) of Fitzgerald Bridge foundation was conducted using a single-pile model with "reference parameters" (reference model, RM) with values of U_{G2} = 0.36m, I_c = 0.6 (in the calculation of F), β_2 = 1/20 and S_r corresponding to the best-fit line shown in Figure 6, as summarized in Table 1. The computed pile displacement and bending moment for the reference model are shown in Figures 7a and 7b, respectively. The displacement at the pile head was 0.21m while the peak bending moment was 9.6 MN-m. The flexural response exceeded the yield level both at the pile head and at the interface between the liquefied layer and underlying base layer.

To illustrate the effects of the uncertainties associated with the liquefied soil and lateral loads on the pile, parametric analyses were conducted in which the parameters listed above were varied within an appropriate range of values, as summarized in Table 1. Results of the analyses are depicted in tornado charts for the peak pile displacement and peak bending moment respectively in Figures 8a and 8b. The response of the reference model (RM) is also indicated in the plots for comparison. The results clearly indicate that the pile response is largely affected by the adopted stiffness and strength of the liquefied soil, and to a lesser extent by the adopted values for U_{G2} and F. In view of the significant effects on the pile response, it is critically important to address these uncertainties through parametric analyses. The unknowns in the pseudo-static analysis are not specific to this approach, but rather they reflect the uncertainties associated with behaviour of piles in liquefying soils. In this context, the pseudo-static analysis as a relatively simple method that captures the essential features of pile behaviour is an excellent tool to address these uncertainties via a systematic parametric study.

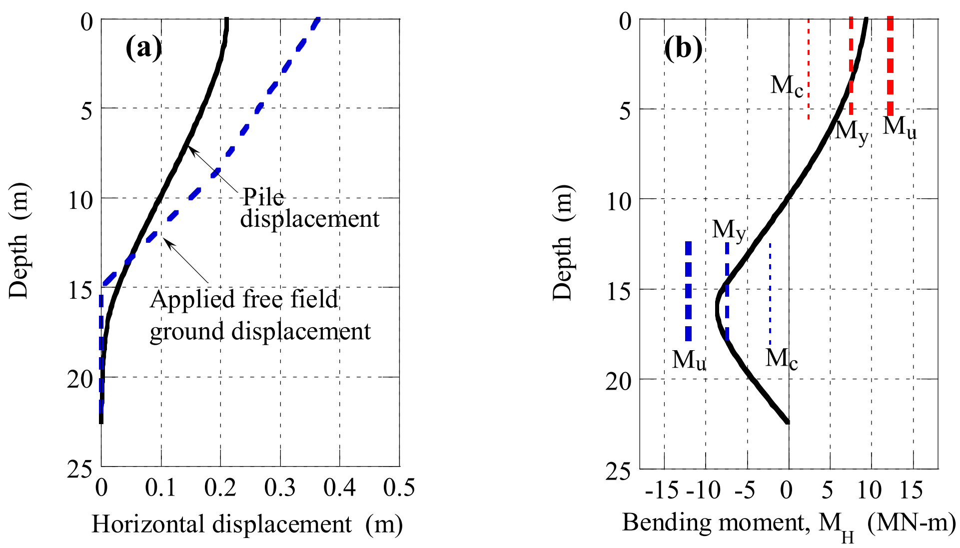


Figure 7: Pile response computed in the pseudo-static analysis using reference model: (a) pile displacement; (b) bending moment

Table 1: Parametric pseudo-static analyses: parameters and results

Parameter varied	Model	Input parameter					Pile Response	
		U_{G2} (m)	I_s	β_2	S_r (kPa)		U_{PH} (m)	M_H (MNm)
					N_1=10	N_1=15		
	RM	0.36	0.6	1/20	14	34	0.21	9.5
U_{G2}	U_{G2}-LB	**0.29**	0.6	1/20	14	34	0.16	8.9
	U_{G2}-UB	**0.43**	0.6	1/20	14	34	0.25	9.9
I_s	I_s-LB	0.36	**0.4**	1/20	14	34	0.18	8.9
	I_s-UB	0.36	**0.8**	1/20	14	34	0.23	10.0
β_2, S_r	β_2, S_r -LB	0.36	0.6	**1/50**	**6**	**22**	0.10	7.8
	β_2, S_r -UB	0.36	0.6	**1/10**	**26**	**48**	0.27	10.3

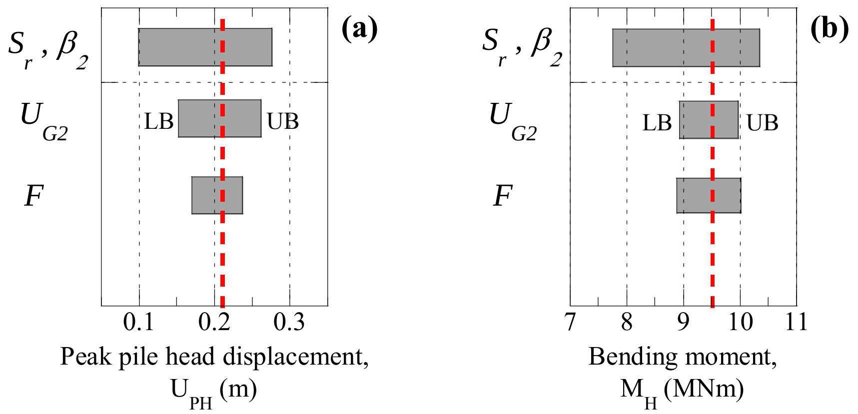


Figure 8: Pile response computed in parametric pseudo-static analyses: (a) pile displacement; (b) bending moment

6 SEISMIC EFFECTIVE STRESS ANALYSIS

Unlike the simplified procedure where the response of the pile is evaluated using a single-pile model and equivalent static loads as input, the seismic effective stress analysis incorporates the soil, foundation and superstructure in a single model and uses an acceleration time history as a base excitation for this model. The seismic soil-structure interaction (SSI) analysis based on the effective stress principle is specifically tailored for analyses of earth structures and soil-structure systems affected by excess pore water pressure and liquefaction, and it is the primary tool for a detailed assessment of liquefaction and its consequences. Key advantages of this analysis procedure are the following:

- The analysis allows detailed simulation of the liquefaction process including build-up of excess pore water pressure, triggering of liquefaction, and subsequent losses in strength and stiffness of liquefied soils. It provides realistic simulation of earthquake loads throughout the depth of the foundation soil by considering responses of individual layers and cross interaction amongst them (base-isolation effects or progressive liquefaction due to upward flow of water).
- Both inertial effects due to vibration of the structure and kinematic effects due to ground movement are concurrently considered while accounting for soil nonlinearity and influence of excess pore pressure on soil behaviour.
- Effects of soil-structure interaction are easily included in the analysis, in which sophisticated nonlinear models can be used both for soils and for structural members. Thus, the analysis permits a rigorous assessment of the seismic performance of the soil-structure system as well as each of its components.

- It allows evaluation of alternative design solutions and effectiveness of countermeasures against liquefaction.

Key disadvantages of the effective stress analysis (and numerical analysis in general) are:
- This analysis imposes high-demands on the user and requires an in-depth understanding of the phenomena considered, the constitutive model used and the theory behind the numerical procedures including the particular software employed. Benchmarking exercises imply that these rigorous requirements are not always satisfied in the profession even when dealing with static problems (Potts, 2003).
- In cases when the analysis is used for a rigorous quantification of the seismic performance of important structures, high-quality geotechnical data from field investigations and laboratory tests are needed.

In author's opinion, the first requirement regarding the knowledge and skills of the user is the key obstacle for a wider use of seismic effective stress analysis in geotechnical practice. Even when conventional data is used as input, this analysis still provides important and unique information on the seismic behaviour of the soil-pile-structure system, as illustrated below.

6.1 Numerical model and modelling of liquefaction resistance

The numerical model for Fitzgerald Bridge used in the effective stress analysis is shown in Figure 9. The model includes the soil, pile foundation and the superstructure. Nonlinear behaviour of the pile was modelled with a hyperbolic moment-curvature *(M-ϕ)* relationship while the soil was modelled using an elastic-plastic constitutive model developed specifically for modelling sand behaviour and liquefaction problems (Cubrinovski and Ishihara, 1998). Details of the modelling will not be discussed herein, but rather the modelling of the liquefaction resistance based on conventional geotechnical data will be demonstrated.

Borelogs, penetration resistance data from CPT and SPT and conventional physical property tests were the only geotechnical data available for the analysis. A rudimentary modelling of stress-strain behaviour of soils considering liquefaction would require knowledge or assumption of the initial stiffness of the soil, strength of the soil and liquefaction resistance. Since none of these were directly available for the soils at this site they were inferred based on the measured penetration resistance. The liquefaction resistance was determined using the conventional procedure for liquefaction evaluation based on empirical SPT charts (Youd et al., 2001). After an appropriate correction for the magnitude of the earthquake, these charts provided the cyclic stress ratio required to cause liquefaction in 15 cycles, which are shown by the solid symbols in Figure 10. Using these values as a target liquefaction resistance, the dilatancy parameter of the model was determined and the liquefaction resistance was simulated for the two layers, as indicated with the lines in Figure 10. Hence, only conventional data was used for determination

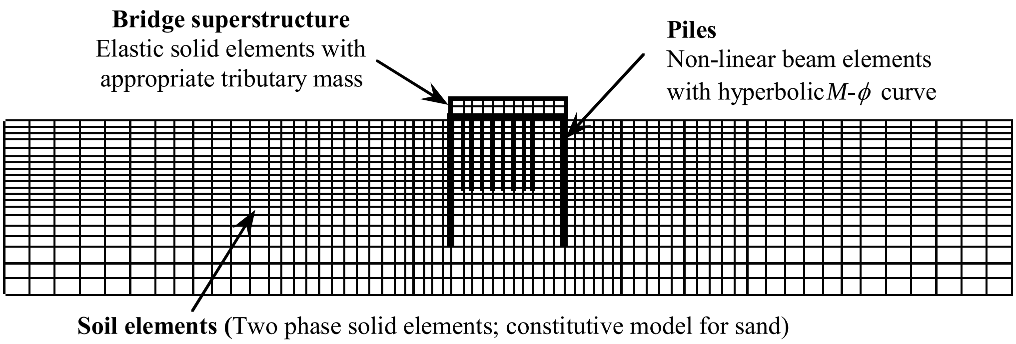


Figure 9: Numerical model used in the seismic effective stress analysis

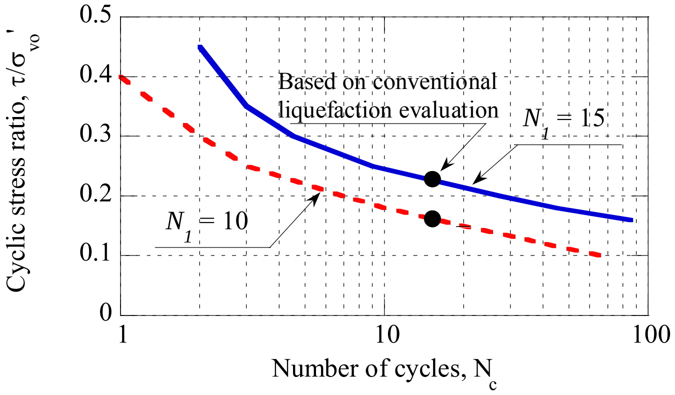


Figure 10: Liquefaction resistance used in the effective stress analysis

of model parameters. While this choice of material parameters practically eliminates the possibility of a rigorous quantification of the seismic response of the soil-pile-structure system, one may argue that the parameters of the model defined as above are at least as consistent and credible as those used in a conventional liquefaction evaluation.

6.2 Results

Figure 11a shows time histories of excess pore water pressure computed at two depths corresponding to the mid depth of layers with N_1 = 10 and N_1 = 15 (z = 13.2m and 7.0m respectively). In the weaker layer, the pore water pressure builds-up rapidly in only one or two load cycles until a complete liquefaction of this layer at approximately 15 seconds. In the denser layer (N_1 = 15), the pore water pressure build up is slower and affected by the liquefaction in the underlying looser layer. The latter is apparent in the reduced rate of pore pressure increase after 15 seconds on the time scale. Clearly, the liquefaction of the loose layer at greater depth produced "base-isolation" effects and curtailed the development of liquefaction in the overlying denser layer. Figure 11b further illustrates the development of the excess pore water pressure throughout the depth of the deposit with time. Note that part of the steady build up of the pore pressure in the upper layer (N_1 = 15) is caused by "progressive liquefaction" or upward flow of water from the underlying liquefied layer. Needless to say, the pore pressure characteristics

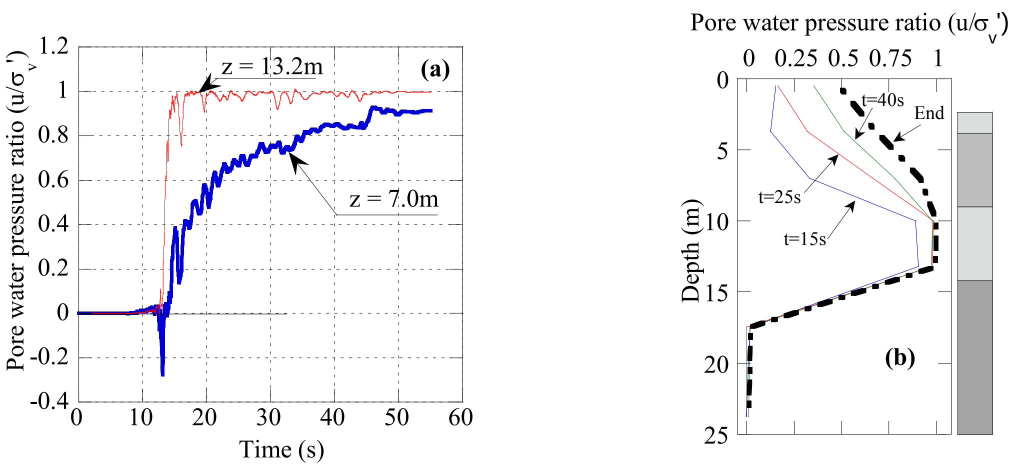


Figure 11: Computed excess pore water pressure: (a) time histories at two different depths; (b) distribution of excess pore water pressures through depth and time

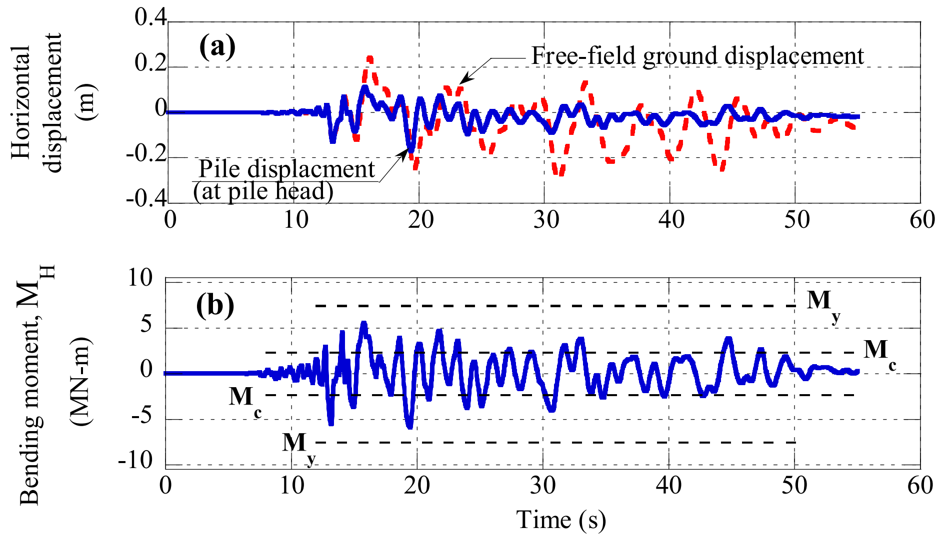


Figure 12: Computed response of the pile: (a) horizontal displacement at pile head; (b) bending moment at pile head

outlined in Figure 11 will be reflected in the shear strain development and lateral displacement of the ground. The seismic effective stress analysis can simulate these important features of the ground response and their effects on the behaviour of the piles and superstructure.

The computed time history of horizontal displacement of the pile is shown in Figure 12a together with the corresponding displacement of the free field soil at the ground surface. The peak pile displacement reached about 0.18m at the top of the pile, which is in agreement with the estimated displacements from the pseudo-static analyses. The response shown in Figure 12a indicates that the peak displacements of the pile and free field soil occurred at different times, at approximately 19 seconds and 32 seconds, respectively. The peak bending moment of the pile was attained at the pile head (M_H) with values slightly below the yield level (Figure 12b). This time history indicates not only the peak level of the response but also the number of significant peaks exceeding cracking level which in turn provides additional information on the damage to the pile. Similar level of detail was available for other components of the numerical model including the foundation soil, old and new piles, and response of the superstructure.

7 PROBABILISTIC APPROACH

A probabilistic approach within the so-called Performance-Based Earthquake Engineering (PBEE) framework has been recently developed for a robust assessment of seismic performance of structures (Cornell and Krawinkler, 2000). This approach employs probabilistic treatment of uncertainties associated with earthquakes, and hence provides an alternative way for assessment of seismic performance of structures. Recently, attempts have been made to expand the application of this approach to geotechnical problems (Kramer, 2008; Ledezma and Bray, 2007; Bradley at. al., 2008). Details of the probabilistic PBEE assessment are beyond the scope of this paper, and instead key features and outcomes of this procedure will be outlined in the following using the case study considered.

7.1 Analysis procedure

Christchurch is located in a region of relatively high seismicity and Fitzgerald Bridge is expected to be excited by a number of earthquakes during its lifespan. Considering all possible

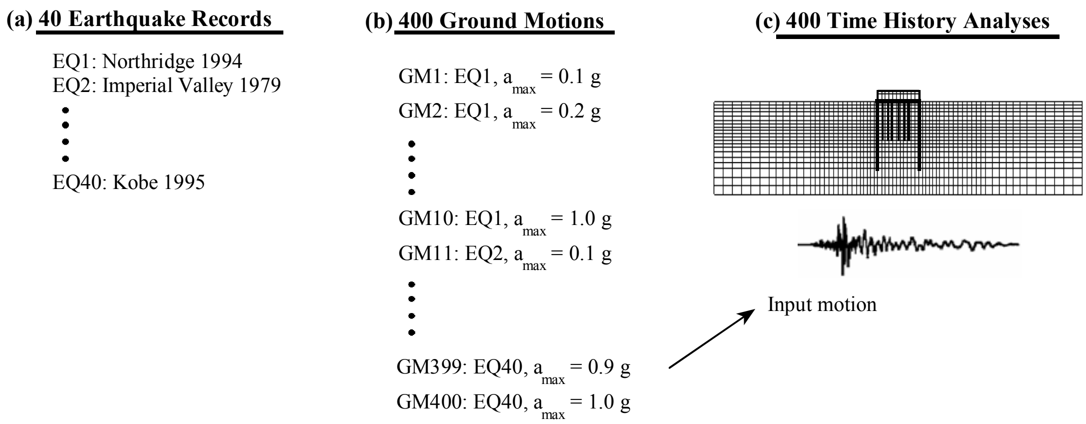


Figure 13: Schematic illustration of multiple analyses used in the probabilistic approach

earthquake scenarios, the response of the bridge and its pile foundation needs to be evaluated for earthquakes with different intensities ranging from very weak and frequent earthquakes to very strong but rare earthquakes. Characteristics of ground motions caused by these earthquakes are very difficult to predict because of the complex and poorly understood source mechanism, propagation of seismic waves and surface-soil effects. In order to account for these uncertainties in the ground motion, the following procedure was adopted.

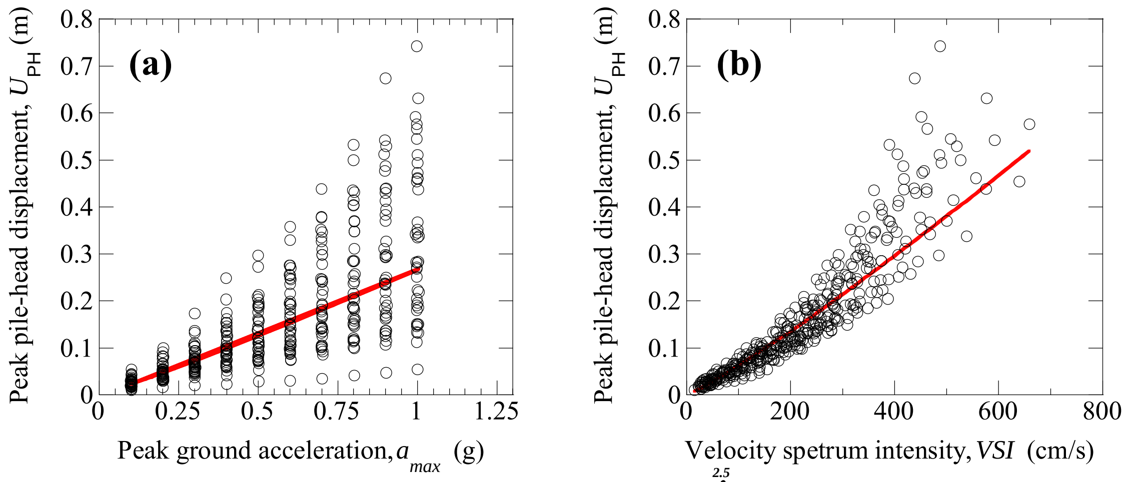
A suite of 40 ground motions recorded during strong earthquakes was first selected, as indicated in Figure 13a. Next, each of these records was scaled to ten different peak amplitude levels, i.e. peak ground accelerations of a_{max} = 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9 and 1.0 g. Thus, 400 different ground motions were created in this way, as indicated in Figure 13b. Using each of these time histories as an input motion, 400 effective stress analyses were conducted using the model shown in Figure 9 and procedures outlined in Section 6, as schematically depicted in Figure 13c.

7.2 Computed response

The next challenge to overcome was how to present the results from 400 time history analyses in a meaningful way. Obviously, some relaxation in the rigorous treatment of response time histories was needed here. This was achieved through the following reasoning:

- Instead of considering the entire time history of the response, the peak amplitude (or other relevant parameter) is used as a measure for the size of the response
- Similarly, a single parameter is used as a measure for the intensity of the ground motion (input time history)
- The results are then presented by correlating the size of the response with the intensity of the ground motion using the abovementioned parameters

For example, one way of presenting the results from the 400 analyses with respect to the pile response is shown in Figure 14a where the peak displacement of the pile head (U_{PH}) computed in the analysis is plotted against the peak acceleration of the input motion (a_{max}). Here, U_{PH} represents a measure for the size of the pile response ("engineering demand parameter", *EDP*) while a_{max} is a measure for the intensity of the ground motion ("intensity measure"). Each open symbol in Figure 14a represents the result (peak response of the pile) from one of the 400 analyses while the solid line is an approximation of the trend from the regression analysis. The scatter of the data is quite large indicating a significant uncertainty in the prediction of the peak response of the pile based on the peak acceleration of the ground motion. Clearly one issue in this approach is the need to identify an efficient intensity measure that reduces the uncertainty and hence improves the predictability of the pile response. However, there is no wide-ranging



$$VSI = \int_{0.1}^{2.5} PSV(T, 5\%)dt$$

$PSV(T, 5\%)$ = *pseudo – velocity spectral ordinate for a period of vibration of T and 5% damping*

Figure 14: Computed pile head displacement (U_{PH}) in 400 effective stress analyses: (a) correlation between (U_{PH}) and a_{max} of input motion; (b) correlation between (U_{PH}) and *VSI* of input motion;

intensity measure that is appropriate for all problems but rather the intensity measure is problem-dependent and is affected by the deformational mechanism and particular features of the phenomena considered. Based on detailed numerical studies, the authors have identified that for piles both in non-liquefiable and liquefiable soils (Bradley et al., 2008) the most efficient intensity measures are velocity-based, and in particular the velocity spectrum intensity, *VSI*. This is illustrated in Figure 14b where the same results for U_{PH} from the 400 analyses shown in Figure 14a are plotted using *VSI* as an intensity measure for the employed input motions. The improved efficiency and predictability of the pile response is evident in the reduced uncertainty as depicted by the smaller dispersion of the data. The plots shown in Figure 14, provide means for estimating the peak response of the piles of Fitzgerald Bridge for all levels of earthquake excitation.

7.3 Assessment of seismic performance: Demand hazard curve

A conventional output from a Probabilistic Seismic Hazard Analysis (PSHA) is the so-called seismic hazard curve which expresses the aggregate seismic hazard at a given site by considering all relevant earthquake sources contributing to the hazard. A seismic hazard curve for Christchurch (Stirling et al., 2001) is shown in Figure 15a where a relationship between the peak ground acceleration (a_{max}) and mean annual rate of exceedance of a given a_{max} is shown. For example, this hazard curve indicates that an earthquake event generating an a_{max} = 0.28g in Christchurch has a recurrence interval or return period of 475 years (or 10% probability of exceedance in 50 years). Key characteristic of the seismic hazard curve is that it combines and rigorously evaluates the uncertainties in the ground motion associated with the earthquake location, earthquake size and variation in the ground motion parameter (e.g. a_{max}) with the distance from the earthquake source.

By combining the seismic hazard curve expressed in terms of a_{max} (Figure 15a) and the correlation between the peak pile response (U_{PH}) and a_{max} established from the results of the effective stress analyses (Figure 15a), a so-called "Demand Hazard Curve" was produced, as shown in Figure 15b. In this way, the probability for exceedance of a certain level of peak pile displacement in any given year (annual rate of exceedance) was estimated for the piles of

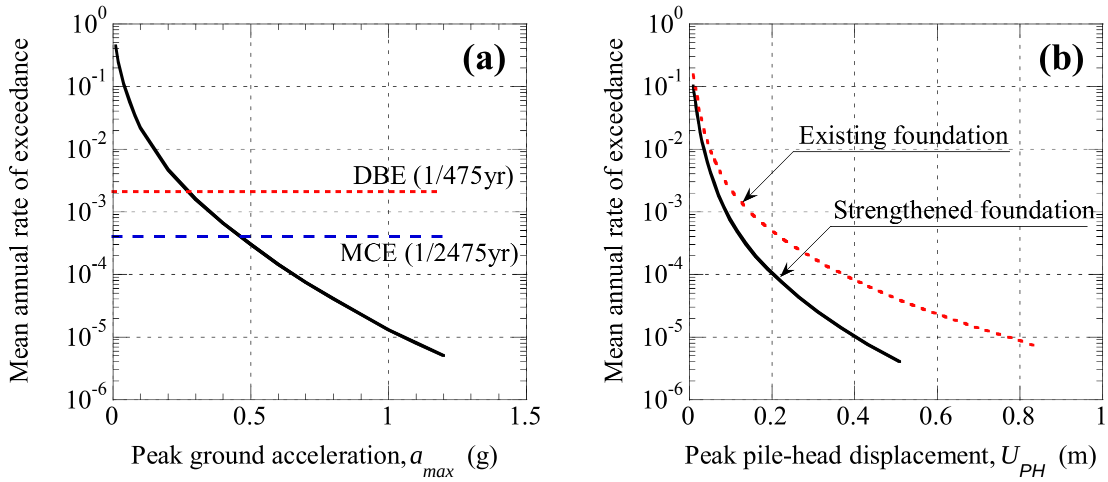


Figure 15: Probabilistic assessment of seismic performance of pile foundation: (a) seismic hazard curve for Christchurch; (b) Demand hazard curve for piles of Fitzgerald Bridge

Fitzgerald Bridge, as shown by the solid line in Figure 14b. A unique feature of the demand hazard curve is that it provides an assessment of the seismic performance of the pile foundation by considering all earthquake scenarios for the site in question.

The peak pile displacement was adopted as a measure for the size of the pile response because it is a good indicator of the damage to the pile (Bradley et al., 2008). Thus, U_{PH} can be converted to a parameter directly correlating with the damage to the pile, e.g. the peak curvature of the pile, and the demand hazard curve can be easily expressed in terms of a damage measure instead of a response measure. Furthermore, the physical damage of the pile foundation will lead to losses, and therefore, the demand hazard curve can be also used to quantify the seismic performance in terms of economic measures (dollars). This in turn will provide an economic basis for decisions on seismic design, repair and retrofit. Clearly, the probabilistic approach provides alternative measures for the performance of the pile while rigorously accounting for the uncertainties associated with the seismic hazard and phenomena considered.

8 SUMMARY AND CONCLUSIONS

Three different approaches for assessment of seismic performance of pile foundations (and soil-structure systems in general) have been presented. These approaches use different models, analysis procedures and are of vastly different complexity. All are consistent with the performance-based design philosophy according to which the seismic performance is assessed using deformational criteria and damage (performance) levels. It is important to emphasize, however, that the three examined approaches focus on different aspects in the assessment of seismic performance and in effect are complementary in nature.

The pseudo-static analysis as a practical approach using conventional geotechnical data and engineering concepts is the most suitable method for assessment of the expected range of deformation and damage to the pile. A simple single-pile model can be used to evaluate the effects of uncertainties associated with behaviour of piles in liquefying soils through systematic parametric analyses. The seismic effective stress analysis, on the other hand, provides very realistic simulation of soil-structure interaction in liquefying soils and importantly allows an integral assessment of the soil-pile-structure system as a whole. Recent experience from strong earthquakes suggest that design concepts in which pile foundations are considered to remain within the elastic range of deformation during strong earthquakes are unrealistic. Hence, there is

Table 2: Methods for assessment of seismic performance of soil-structure systems: key features and contributions

Method of assessment	Key feature	Specific contributions in the assessment
Pseudo-static analysis	• Simple • Conventional data and engineering parameters	• Evaluates the response and damage level for a single pile while taking into account relevant uncertainties • Enhances foundation design
Seismic effective stress analysis	Realistic simulation of soil-structure interaction in liquefying soils	• Detailed assessment of seismic response of pile foundations including effects of liquefaction and SSI • Integral assessment of inelastic behaviour of pile-foundation-structure systems • Enhances communication between geotechnical and structural engineers
Probabilistic PBEE framework	• Considers all earthquake scenarios • Quantifies seismic risk	• Addresses uncertainties associated with ground motion characteristics on a site specific basis • Provides engineering measures (response and damage) and economic measures (losses) of performance • Enhances communication of design outside profession

a need to consider inelastic deformation concurrently in both the superstructure and pile foundation. Advanced seismic analyses provide this capability and methods based on the effective stress principle further permit the inclusion of effects of excess pore pressures and liquefaction in the assessment. Finally, the probabilistic approach offers a unique perspective in the assessment of seismic performance, first through a rigorous treatment of the single most important source of uncertainty in seismic studies, the ground motion, and then by providing alternative performance measures in the assessment, engineering and economic ones. Key features and specific contributions of the examined approaches are summarized in Table 2. All of these analysis procedures improve our understanding of complex seismic behaviour and enhance engineering judgement, which after all, is probably the most significant contribution that one can expect from such an exercise.

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