Project BIE 08/545:

ANALYSIS AND DESIGN OF PILES IN LIQUEFYING SOILS

Report prepared for the New Zealand Earthquake Commission (EQC)

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April 2010

EXECUTIVE SUMMARY

The pseudo-static method of analysis (PSA) is a simplified design-oriented approach for analysis of seismic problems based on routine computations and conventional engineering models. The application of the method to analysis of piles in liquefying soils is burdened by significant uncertainties associated with soil liquefaction, soil-pile interaction in liquefying soils and the need to reduce a very complex dynamic problem to a simple equivalent static analogy. Hence, despite its simplicity, the application of the pseudo-static analysis is not straightforward and requires careful consideration of the uncertainties in the analysis. This study addresses some of the key issues that arise in the application of the pseudo-static analysis to piles in liquefying soils, and makes progress towards the development of a clear modelling (analysis) strategy that permits a consistent and reliable use of the simplified pseudo-static analysis.

Characteristics of liquefying soils and loads on piles are significantly different during the cyclic phase (strong ground shaking) and in the subsequent lateral spreading phase, and therefore, it is necessary to separately consider these two phases in the simplified analysis of piles. This paper describes a practical PSA procedure for preliminary assessment and design of piles, and addresses key parameters and uncertainties in the analysis, both for the cyclic phase and the lateral spreading phase of the pile response.

A comprehensive parametric study was conducted in which a wide range of soil-pile systems, loading conditions and values for model parameters in the PSA were considered, for piles in laterally spreading soils. Results from the analyses were used to examine and quantify the sensitivity of the pile response to various model parameters, and to establish a fundamental link between the sensitivity of the pile response and the mechanism of soil-pile interaction. On this basis, some general principles for conducting pseudo-static analysis of piles in liquefying soils could be established that apply across-board to different soil-pile systems and loading conditions.

In the simplified pseudo-static analysis of piles, the ultimate lateral pressure from the liquefied soil is commonly approximated based on the residual strength of liquefied soils. This strength does not have sound theoretical basis, but rather is estimated from one of several empirical relationships between the residual strength and penetration resistance. The two empirical relationships adopted in this study, even though originating from the same database, result in substantially different strength profiles (ultimate lateral pressures on the pile) throughout the depth of the liquefied layer. Series of analyses were conducted to investigate the effects of strength normalisation on the pile response predicted by the pseudo-static analysis. It is shown that effects of

strength normalisation can be quite significant and that they depend on the relative stiffness of the pile and the thickness of a non-liquefiable crust at the ground surface.

The cyclic study comprises two distinct phases, the first considering the response of piles when subjected only to cyclic soil displacements, and the second considering the pile response when both cyclic soil displacements and superstructure inertial forces are present and acting simultaneously. In this comprehensive series of analyses emphasis was placed on understanding the governing mechanisms and controlling-parameters when simultaneously considering the combined inertial loads from the superstructure and kinematic loads due to lateral ground movements in PSA.

Finally, three different approaches for assessment of seismic performance of piles in liquefying soils comparatively examined. 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. Even though the methods nominally have the same objective, it is shown that they focus on different aspects in the assessment and provide alternative performance measures. In this context, key features of the PSA and its unique contribution in the assessment of pile foundations in liquefying soils is discussed.

PLAIN ENGLISH SUMMARY

During the intense shaking that accompanies large earthquakes groundwater pressures can raise causing soil to lose some of its strength and capacity to support structures resting on it. In the extreme case, the soil may lose its strength completely and the resulting "soil liquefaction" may cause severe damage to buildings, bridges and other engineering structures. Pile foundations are especially vulnerable to liquefaction since they are used to support structures near (in) river beds, in reclaimed land and coastal areas that are susceptible to liquefaction. This issue is very relevant for New Zealand since liquefaction is recognized as one of the principal seismic hazards affecting urban centres as well as critical lifelines and infrastructure across the country.

The processes of soil liquefaction and "soil-pile interaction" during earthquakes are extremely complex. Hence, the design of piles against earthquake loads and soil liquefaction is a very difficult task. There are several methods available to engineers for seismic design and analysis of piles. The most attractive approach for the profession is the "pseudo-static analysis", because it is relatively easy to understand and use despite the complexity of the processes that are considered in the analysis. The pseudo static method of analysis is routinely used in the engineering practice and is commonly stipulated in modern seismic design codes.

One of the key issues in the application of the pseudo-static analysis arises from the uncertainties associated with liquefaction and unknowns in the analysis. This is not surprising in view of the fact that a very simple (static) method is used as a basis for modelling very complex (dynamic) problem. Hence, it is difficult for the designer to figure out how to model the complex processes with the simple analysis, and to know whether the adopted assumptions are on the safe side or not. Very little guidance exists in the profession in this regard.

The research presented in this report aims at providing clear guidance how to use the pseudo-static method for analysis of piles in liquefying soils. It shows which model parameters are the most important in the analysis and provides analysis strategy to the designer. The study is based on observations of the performance of pile foundations in recent strong earthquakes, benchmark experimental studies and comprehensive analytical studies conducted over the past ten years. It aims at developing simple yet effective procedure for analysis and design of piles in liquefying soils, considering specific New Zealand conditions.

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CHAPTER 1: INTRODUCTION

1.1 PROJECT OBJECTIVES

This report summarizes the output from a two-year research conducted at the University of Canterbury within the Biennial Research Project BIE08/545 “Analysis and Design of Piles in Liquefying Soils”, funded by EQC. Associate Professor Misko Cubrinovski was the project leader and principal investigator, while Jennifer Haskell (former undergraduate and research student) and Dr. Brendon Bradley (former PhD student; currently continuing staff at UC) were investigators on the project.

The principal objective of the research was to further develop and improve a simplified method for pseudo-static analysis of piles in liquefying soils that has been originally developed by the principal investigator over the past 10 years. One of the key issues in the analysis of piles in liquefying soils is how to deal with the uncertainties associated with the soil-pile interaction in liquefied soils. The issue is particularly significant because the parameters affected by the uncertainties and unknowns in the analysis essentially control the seismic performance of pile foundations. The presented research effort aimed at addressing the most relevant issues in the analysis of piles in order to further develop the method and establish a sound basis for its incorporation in the New Zealand practice. The project focussed on the following specific objectives:

- To identify key model parameters and critical uncertainties in the simplified pseudo-static analysis of piles in liquefying soils *(in order to employ appropriate modelling strategies and focus the attention in the analysis on issues that matter the most).*
- To quantify the effects of uncertainties and variation in model parameters on the response of piles subjected to lateral spreading *(in order to identify the most important model parameters and quantify their effects on the pile response in relation to the deformation mechanism and stage of loading).*
- To quantify the effects of shear strength normalisation on the response of piles predicted by the pseudo-static analysis *(to answer the question, do we need to use a normalised soil strength in the calculation of the ultimate soil pressure or not*).
- To quantify the effects of modelling uncertainties on the cyclic response of piles in liquefying soils *(or determine how to combine in a static analysis the kinematic effects due to ground movement and inertial effects due to vibration of the superstructure).*

1.2 ORGANIZATION OF THE REPORT

This report is a collection of chapters which are stand-alone publications that have either been published over the past two years or are currently in print. All publications except that comprising Chapter 5 have already undergone a peer review. For this reason, some repetition does occur especially in the introductory parts of the chapters where general features of the problem and the adopted methodology are described. Also, the conclusions only refer to issues specifically addressed in a given chapter.

Chapter 2 introduces the problem and describes key features of soil-pile interaction in liquefying soils using case histories from past earthquakes and benchmark full-size experiments. It also outlines the problems that a simplified pseudo-static analysis encounters in the context of the complex phenomena considered and, in particular, the uncertainties and unknowns in the analysis.

In *Chapter 3,* results from a comprehensive parametric study are presented in order to examine and quantify the sensitivity of the pile response to various model parameters. A wide range of soil-pile systems, loading conditions and values for model parameters are considered aiming to develop a set of general rules for conducting pseudo-static analysis of piles in liquefying soils. Results form the analyses clearly depict a various sensitivity of the pile response to different parameters of the model, and relate it to the particular deformation mechanism (relative displacements between the pile and the soil) or stage of loading (magnitude of lateral ground movement). The outcome of this work provides a sound basis for establishing a hierarchy amongst different model parameters and developing an efficient strategy in the application of the pseudo-static analysis which is essential in view of the significant uncertainties and unknowns in the analysis.

Chapter 4 focuses on a particular modelling issue whether to normalise the shear strength or not when calculating the ultimate lateral soil pressure on the pile. The study investigates the effects of this normalisation on the response of the pile predicted by the simplified pseudo-static analysis and provides guidance on the size of these effects and situations at which they are significant.

In *Chapter 5,* results from another comprehensive parametric study are presented which focuses on the cyclic phase of the pile response in liquefying soils. Again a wide range of soil-pile systems, loading conditions and values for model parameters are considered in order to investigate the sensitivity of the cyclic response of the pile to different model parameters. Particular attention is given to the question how to combine the kinematic loads due to ground movement and inertial effects due to vibration of the superstructure in the simplified pseudo-static analysis.

Finally, *Chapter 6* comparatively examines the pseudo-static method of analysis with two alternative and more sophisticated approaches (dynamic time history analysis, and the probabilistic assessment) and points out that, even though these methods have nominally the same objective, they focus on different aspects and make different contribution in the assessment of the seismic performance of pile foundations.

CHAPTER 2: PSEUDO-STATIC ANALYSIS OF PILES SUBJECTED TO LATERAL SPREADING

Cubrinovski, M., Ishihara K. and Poulos H. (2009). Special Issue, Bulletin of the NZ Society for Earthquake Engineering, Vol. 42, No. 1, 28-38.

ABSTRACT

Soil liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness in the liquefied soil, and consequent large lateral ground movements. Both cyclic displacements during the intense ground shaking and development of liquefaction, and especially post-liquefaction displacements due to spreading of liquefied soils are damaging for piles. Characteristics of the liquefied soils and loads on piles are significantly different during the cyclic phase and in the subsequent lateral spreading phase, and therefore, it is necessary to separately consider these two phases in the simplified analysis of piles. This paper describes a practical procedure for preliminary assessment and design of piles subjected to lateral spreading, and addresses key parameters and uncertainties involved.

2.1 INTRODUCTION

There are several methods available for analysis of piles in liquefied soils including sophisticated finite element analysis based on the effective stress principle and simplified methods based on the pseudo-static approach. A rigorous effective stress analysis permits evaluation of seismic soil-pile interaction while considering the effects of excess pore pressure and eventual soil liquefaction on the pile response. Whereas the predictive capacity of such analysis has been verified in many studies, its application in engineering practice is constrained by two requirements, namely, the required high-quality and specific data on the in-situ conditions, physical properties and mechanical behaviour of soils, and quite high demands on the user regarding the knowledge and understanding both of the phenomena considered and particular features in the adopted numerical procedure. Provided that the above requirements are met, however, the effective stress analysis provides an excellent tool for assessment of the seismic performance of pile foundations in liquefiable soils.

For preliminary assessment and design of piles, however, a simplified analysis may be more appropriate provided that such analysis can satisfy the following requirements: i) the adopted model must capture the kinematic mechanism associated with the spreading of liquefied soils; ii) The analysis should allow us to estimate the inelastic response and damage to piles, and iii) the method should allow for variations in key parameters and assessment of the uncertainties associated with lateral spreading. Based on these premises, this paper addresses the use of the pseudo-static analysis of piles in liquefying soils and focuses in particular on its application to the analysis of piles subjected to lateral spreading.

2.2 GROUND DISPLACEMENTS IN LIQUEFIED SOILS

When analyzing the behaviour of piles in liquefied soils, it is useful to distinguish between two different phases in the soil-pile interaction: a cyclic phase in the course of the intense ground shaking and consequent development of liquefaction, and a lateral spreading phase following the liquefaction. During the cyclic phase, the piles are subjected to cyclic horizontal loads due to ground displacements (kinematic loads) and inertial loads from the superstructure, and the combination of these oscillatory kinematic and inertial loads determines the critical load for the integrity of the pile during the shaking. Lateral spreading, on the other hand, is primarily a post-liquefaction phenomenon that is characterized by very large unilateral ground displacements and relatively small inertial effects. Thus, the liquefaction characteristics and lateral loads on piles can be quite different between the cyclic phase and the subsequent lateral spreading phase.

Cyclic Ground Displacements

In order to illustrate some important features of ground displacements in liquefied soils, observations from well documented case histories in the 1995 Kobe earthquake are discussed in the following. Figures 1a and 1b show computed horizontal ground displacements and excess pore water pressures that developed in an 18 m thick fill deposit during the intense part of the ground shaking in this quake. This response is representative of the cyclic phase of the free field response of the fill deposits in areas that were not affected by lateral spreading.

Several features of the ground response shown in Figure 1 are relevant to the behaviour and analysis of piles in liquefied soils. First, the cyclic horizontal ground displacements in the course of the strong shaking are very large with peak values of about 35-40 cm. These displacements correspond to an average peak shear strain of about 3-4 % throughout the 10-12 m depth of the liquefied layer. Next, it is important to note that at the time when the ground displacement reached a large value of about 30 cm for the first time since the start of the shaking, i.e. at approximately 5.3 sec, the excess pore water pressure was well below the effective overburden stress thus indicating that the soil has not fully liquefied, at this stage. These large displacements were accompanied with high ground accelerations of about 0.4 g at the ground surface. This type of behaviour, where large ground displacements and high accelerations concurrently occur just before or at the time of development of full liquefaction, has been also observed in shake table experiments, thus highlighting the need to carefully consider the combination of inertial loads from the superstructure and kinematic loads due to ground displacements when analyzing the behaviour of piles during the cyclic phase. The magnitude of these loads depends on a number of factors including the excess pore water pressure build-up, relative displacements between the soil and the pile, and relative predominant periods of the ground and superstructure, among others. Clear and simple rules for combining the ground displacements (kinematic loads) and inertial loads from the superstructure in the simplified pseudo-static analysis have not been established yet, though some suggestions may be found in Tamura and Tokimatsu (2005) and Liyanapathirana and Poulos (2005).

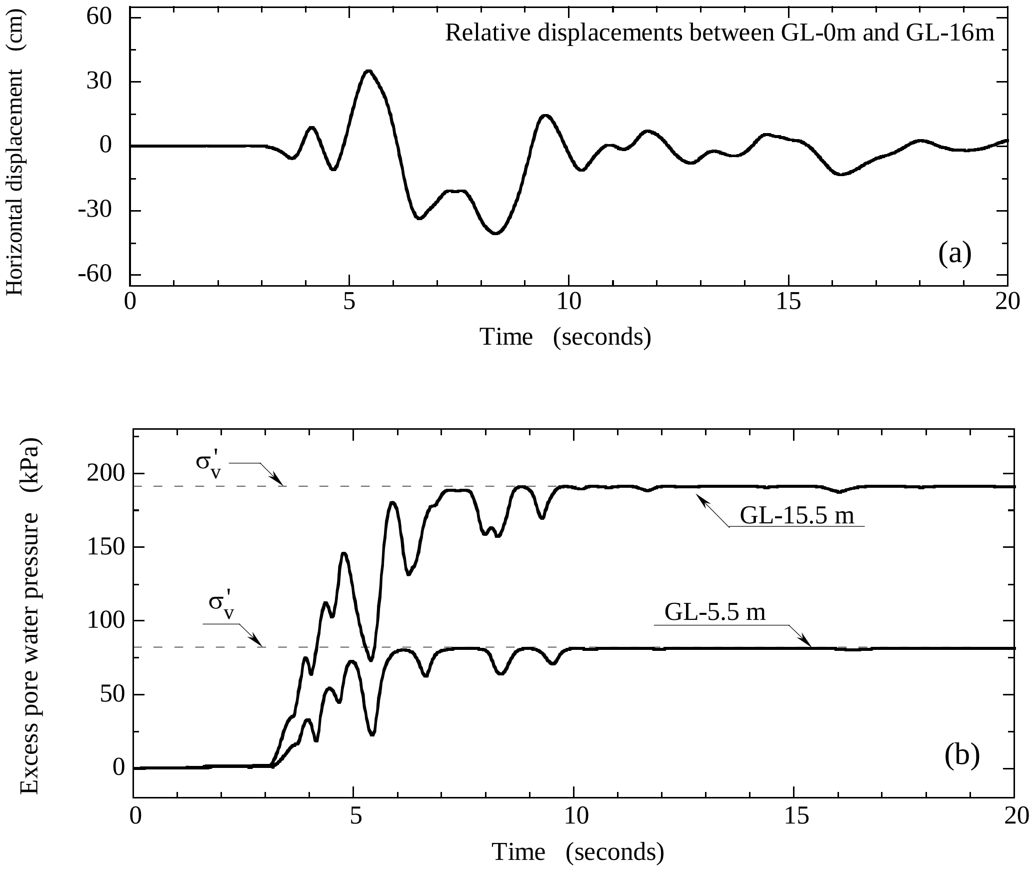

Figure 1. *Ground response of liquefied deposit in the 1995 Kobe earthquake: (a) Cyclic ground displacement; (b) Excess pore water pressure*

Lateral Spreading Displacements

In the 1995 Kobe earthquake, the ground distortion was particularly excessive in the waterfront area where many quay walls moved several meters towards the sea and lateral spreading occurred in the backfills that progressed inland as far as 200 m from the revetment line. Ishihara et al. (1997) investigated the features of movements of the quay walls and ground distortion in the backfills by the method of ground surveying and summarized the measured displacements in plots depicting the permanent ground displacement as a function of the distance inland from the waterfront, as shown in Figure 2. Here, the shaded area shows the range of measured displacements along N-S sections of Port Island, and the solid line is an approximation for the average displacement. Superimposed in Figure 2 are the cyclic ground displacements in the free field showing that in the zone within a distance of approximately 50 m from the quay walls, the permanent ground displacements due to lateral spreading were significantly greater than the cyclic ground

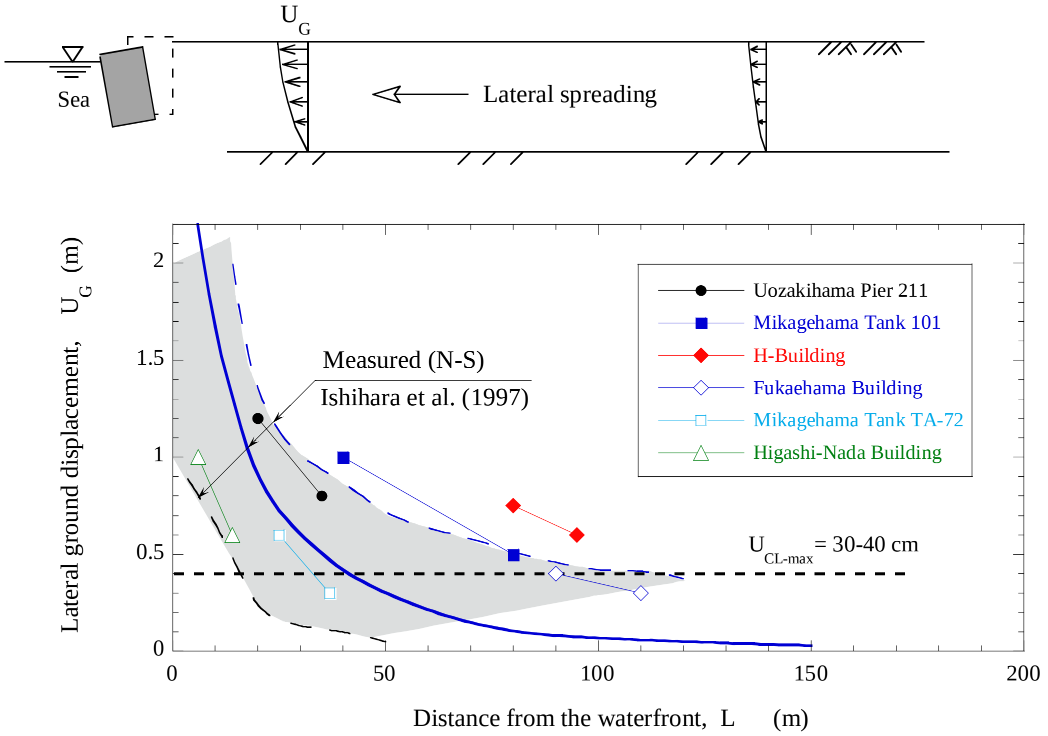

Figure 2. Permanent lateral ground displacements due to spreading of liquefied soils in the 1995 Kobe earthquake

displacements. The permanent ground displacements reached about 1 - 4 m at the quay walls. Since the lateral spreading is basically a post-liquefaction phenomenon, it is associated with higher excess pore pressures and hence lower stiffness of the liquefied soils, as compared to its preceding cyclic phase. This feature, together with the unilateral down-slope or seaward ground movement, results in very large permanent spreading displacements. Clearly the magnitude and spatial distribution of ground displacements, as well as the stiffness of the soils undergoing large lateral movements, are quite different between the cyclic phase and lateral spreading phase, and these differences have to be accounted for in the simplified analysis of piles.

2.3 TYPICAL DAMAGE TO PILES

A large number of pile foundations of buildings, storage tanks and bridge piers located in the waterfront area of Kobe were damaged in the 1995 Kobe earthquake (Ishihara and Cubrinovski, 1998; 2004; JGS, 1998; Tokimatsu and Asaka, 1998). Detailed field investigations were conducted on selected piles using a borehole video camera and inclinometers for inspecting the crack distribution and deformation of the pile respectively throughout the depth of the deposit, as well as by a visual inspection of the damage to the pile head. By and large, the damage to the piles can be summarized as follows:

1. Most of the piles suffered largest damage at the pile top and in the zone of the interface between the liquefied layer and the underlying non-liquefied layer (Figure 3).

2. Piles in the zone of large lateral spreading displacements were consistently damaged at depths corresponding to the interface between the liquefied layer and the underlying non-liquefied layer. Since this interface was at large depths where inertial effects from the superstructure are known to be less significant, this damage can be attributed to the lateral loads arising from the excessive ground movement due to spreading.
3. Damage at the pile head was encountered both for piles in the free field and piles located within the lateral spreading zone, near the quay walls. Both inertial loads from the superstructure and kinematic loads due to lateral ground displacements contributed to the damage at the pile head.
4. The variation of lateral spreading displacements with the distance from the waterfront shown in Figure 2 may result in different lateral loads being applied to individual piles, depending on their position within the pile-group. This in turn may lead to significant cross-interaction effects and consequent bending deformation and damage to piles in accordance with these interaction loads from the pile-cap-pile system. In some cases where these pile-group effects were significant, the piles failed within the liquefied layer or at least several meters below the pile head.

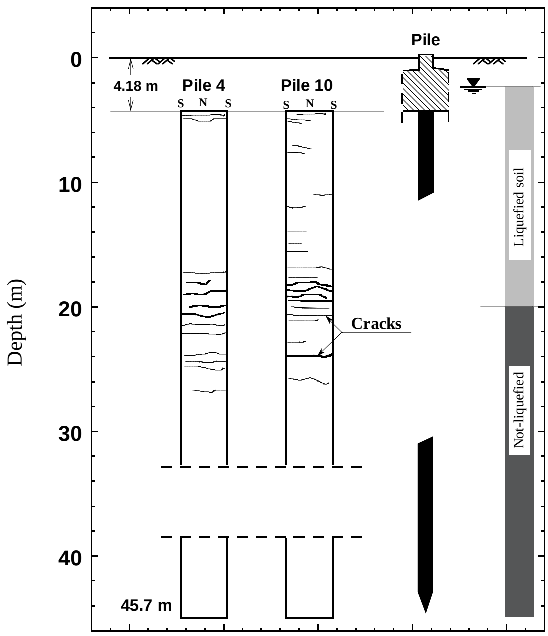

Figure 3. Typical damage to piles observed in the 1995 Kobe earthquake (Uozakihama bridge pier P211)

In addition to the typical damage patterns described above, other less significant damage was consistently found at various depths for many of the inspected piles thus reflecting the complex dynamic nature of loads and behaviour of piles in liquefying soils.

2.4 PSEUDO-STATIC APPROACH FOR SIMPLIFIED ANALYSIS

The most frequently encountered soil profile for piles in liquefied deposits consists of three distinct layers, as illustrated in Figure 4 where the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and non-liquefied base layer. Liquefaction during strong ground shaking results in almost a complete loss of strength and stiffness of the liquefied soil, and consequent large lateral ground displacements. As demonstrated in the previous section, particularly large and damaging for piles are post-liquefaction displacements due to lateral spreading. During spreading, the non-liquefied surface layer is carried along with the underlying spreading soil, and when driven against embedded piles, the crust layer is envisioned to exert large lateral loads on the piles. Thus, the excessive lateral movement of the liquefied soil, lateral loads from the surface layer and significant stiffness reduction in the liquefied layer, are key features that need to be considered when evaluating the pile response to lateral spreading.

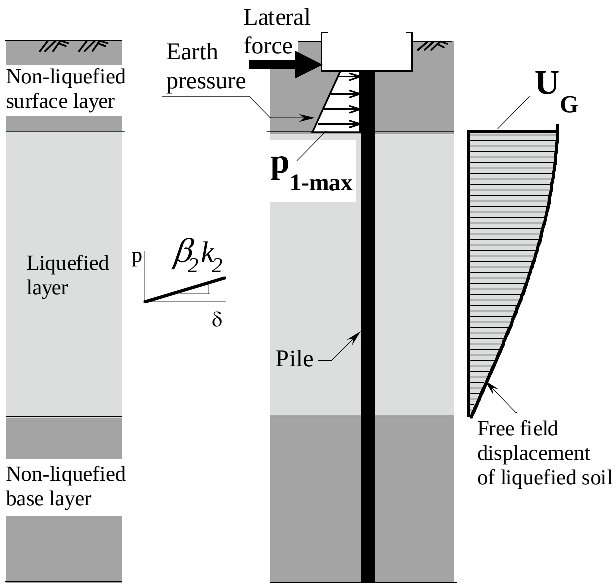

Figure 4. Simplified kinematic mechanism of lateral spreading

In light of the liquefaction characteristics and kinematic mechanism as described above, a three-layer soil model was adopted for a simplified analysis of piles based on the pseudo-static approach, in a previous study (Cubrinovski and Ishihara, 2004). As indicated in Figure 4, in this model the pile is represented by a continuous beam while the interaction between the liquefied soil and the pile (p-δ relationship) is specified by an equivalent linear spring ($\beta_2 k_2$). Here, k_2 is the subgrade reaction coefficient while β_2 is a scaling factor representing the degradation of stiffness due to liquefaction and nonlinear behaviour. In the analysis, the spreading is represented by a horizontal free-field displacement of the liquefied soil while effects of the surface layer are modelled by an earth pressure and lateral force at the pile head, as illustrated in Figure 4. Using an iterative procedure based on the equivalent linear approach, a closed-form solution was developed for evaluating the pile response to lateral spreading. The analysis permits estimation of the inelastic response and damage to piles, yet it is based on a simple model that requires a small number of conventional engineering parameters as input (Figure 5). Needless to say, one may use an FEM beam-spring model instead of the above closed-form solution and conduct even more rigorous analysis, as compared to the three-layer model, because it will permit consideration of a multi-layer deposit with different load-deformation properties. In principle, however, the following discussion applies to the pseudo-static analysis of piles, in general.

Input parameters of the computational model and adopted load-deformation relationships for the soil and the pile are shown in Figure 5. The equivalent linear p-δ relationship for the liquefied layer was adopted in order to simplify the modelling of the highly nonlinear behaviour of liquefied soils undergoing spreading and to allow parametric evaluation of the effects of this parameter. In the analysis of a given pile, it is envisioned that β_2 will serve as a parameter that will be varied over a relevant range of values, thus permitting evaluation of the pile response by assuming different properties of the liquefied soil. On the other hand, bilinear p-δ relationships and a tri-linear moment-curvature relationship (M-ϕ) were adopted for modelling the nonlinear behaviour of the non-liquefied soil layers and the pile respectively. Note that p_{1-max} defines the ultimate lateral pressure that can be applied by the crust layer to the pile. In cases when the relative displacements between the soil and the pile are very large, it would be necessary to limit the maximum pressure that the liquefied soil can apply to the pile. The residual strength of liquefied soils would be one obvious choice in the definition of the ultimate pressure from the liquefied soil on the pile. This modification of the original model is indicated with a dashed line in the p-δ relationship for the liquefied soil in Figure 5.

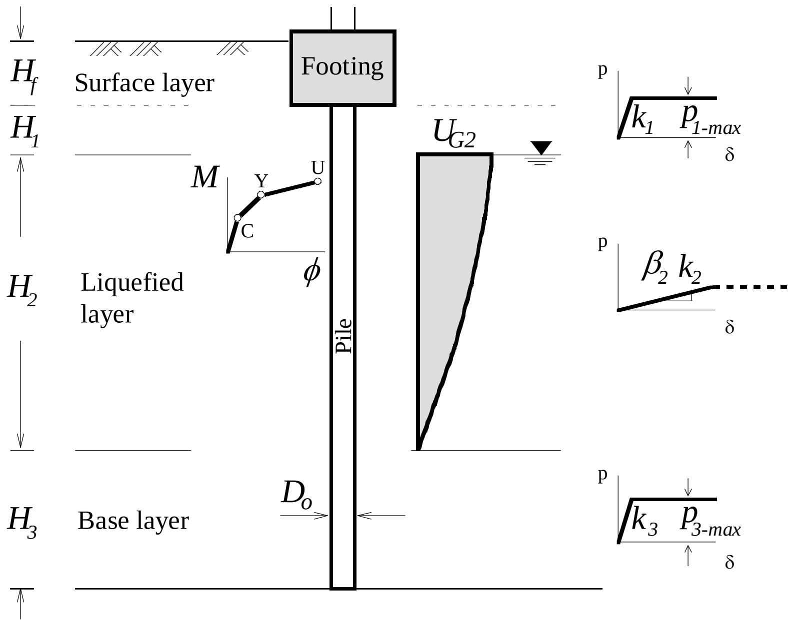

Figure 5. Characterization of nonlinear behaviour and input parameters of the model

2.5 KEY PARAMETERS AND UNCERTAINTIES INVOLVED

The analysis of piles in liquefying soils is burdened by unknowns and uncertainties associated with liquefaction and lateral spreading in particular. Thus, it is very difficult to estimate the strength and stiffness properties of liquefied soils or predict the magnitude and spatial distribution of lateral spreading displacements. One of the key aspects of the simplified analysis is therefore to properly address these uncertainties.

Lateral Ground Displacements

The lateral displacement of the spreading soil (U_{G2}) can be evaluated using empirical correlations for ground displacements of lateral spreads (Ishihara et al., 1997; Tokimatsu and Asaka, 1998; Hamada et al., 2001; Youd et al., 2002). It is important to recognize, however, that in most cases it would be very difficult to make a reliable prediction for the spreading displacements. This difficulty is well illustrated in Figure 2 where a large scatter in the ground displacements is seen, even for a single earthquake event and generally similar ground conditions. In this context, Youd et al. (2002) suggested the use of a factor of 2 for the displacements predicted with their empirical model, in order to cover the expected range of variation in the spreading displacements.

Cyclic ground displacements can be estimated more accurately by means of an effective stress analysis, but this combination of an advanced analysis being used for the definition of the input in a simplified analysis is not consistent or practical. For this reason, it seems more appropriate to estimate the peak cyclic displacements for the simplified analysis by using simplified charts correlating the maximum cyclic shear strain that will develop in the liquefied layer with the cyclic stress ration and SPT blow count, as suggested by Tokimatsu and Asaka (1998), for example. The horizontal cyclic displacement profile can be then easily obtained by integrating the shear strains throughout the depth of the liquefied layer. In both cases of cyclic liquefaction and lateral spreading, the lateral ground displacement that is used as an input in the simplified analysis of piles is a free field ground displacement which is unaffected by the pile foundation.

Crust Layer

The lateral load from the non-liquefied crust layer may often be the critical load for the integrity of the pile because of its large magnitude and unfavourable position as“top-heavy” load acting above a laterally unsupported portion of the pile in the liquefied soil. For the adopted bilinear p-δ relationship for the crust layer shown in Figure 5, the key input parameter is the ultimate lateral pressure, p_{1-max}.

The ultimate soil pressure from the surface layer per unit width of the pile can be estimated using a simplified expression such as, $p_{1-max} = \alpha_u \, p_p$, where $p_p(z_1)$ is the Rankine passive pressure while α_u is a scaling factor to account for the difference in the lateral pressure between a single pile and an equivalent wall. Figure 6 shows the variation of α_u with the relative displacement observed in a benchmark lateral spreading experiment on full-size piles (Cubrinovski et al., 2006) with the maximum lateral pressure on the single pile being about 4.5 times the Rankine passive pressure. Data from other experimental studies, shown in Figure 7, also indicate quite large values for the parameter α_u, clearly indicating that very large lateral loads can be applied by the crust layer to the pile. Here, it is important to distinguish between two types of loading conditions, namely, active pile loading and passive pile loading. In the case of active pile loading, the horizontal force at the pile is the causative load for the pile deformation, as shown in Figure 8a; in this case, the mobilized earth pressure provides the resisting force. In the case of passive pile loading, on the other hand, the mobilized pressure from the crust layer provides the driving force for the pile deformation, as illustrated in Figure 8b. Note that in Figure 7 the two sets of experimental data on passive piles yield a value of α_u = 4.5. The test data used by Broms (1964) yielded mostly values of α_u = 3 – 6, and Broms adopted the lower-bound value of α_u = 3 as a conservative estimate for active piles. This value has been adopted in many design codes for active loading on piles, but may be unconservative for passive piles.

It is important to note in Figure 6 that a large relative displacement of nearly 20 cm was needed to mobilize the ultimate lateral pressure from the crust layer. This relative displacement δ_u at which p_u is mobilized depends on the relative density of the sand, as illustrated by the experimental data summarized in Figure 9. Here H denotes the height of the model wall or pile cap used in the test. It is evident that for dense sands with D_r = 70 % to 80 %, the ultimate pressure was mobilized at a relative displacement of about δ_u = 0.02H to 0.08H and that larger movement was needed to mobilize the passive pressure in loose sand. Rollins (2002) suggested that the presence of a low strength layer below the surface layer may increase the required deflection to mobilize the passive pressure, and this appears to be a relevant observation for a crust layer overlying liquefied soils.

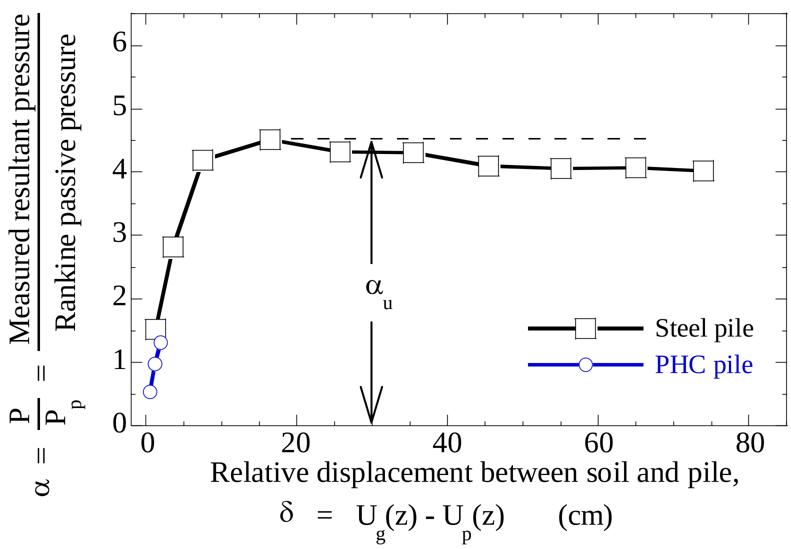

Figure 6. *Ratio of lateral pressure from the crust layer on a single pile and Rankine passive pressure, measured in full-size test using large-scale shake table*

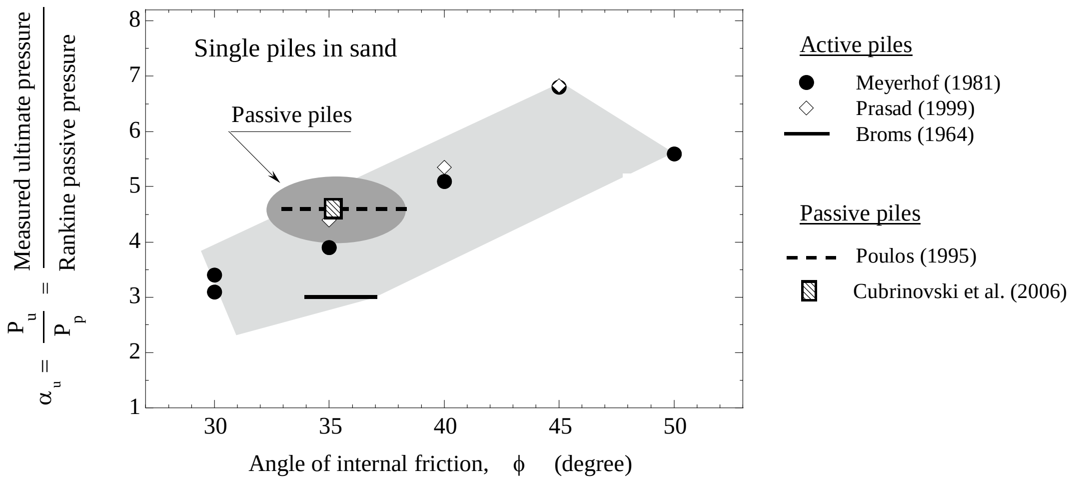

Figure 7. Ratio of ultimate pressure from the crust layer on a single pile and Rankine passive pressure, obtained in experimental studies

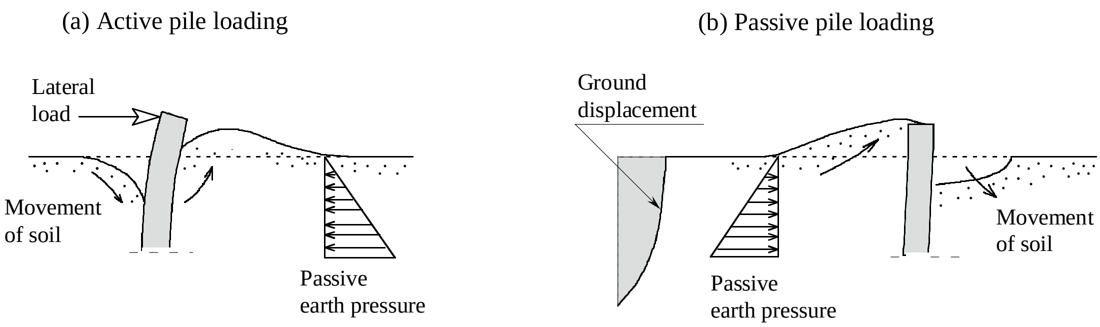

Figure 8. Schematic illustration of lateral loading of piles: (a) Active-pile-loading; (b) Passive pile-loading

Liquefied Layer

The factor β_2, which specifies the reduction of stiffness due to liquefaction and nonlinear behaviour ($\beta_2 k_2$), is affected by a number of factors including the density of sand, excess pore pressures, magnitude and rate of ground displacements, and drainage conditions. Typically, β_2 takes values in the range between 1/50 and 1/10 for cyclic liquefaction and between 1/1000 and 1/50 in the case of lateral spreading. The values of β_2 back-calculated from full-size tests on piles (Cubrinovski et al., 2006) are shown in Figure 10 as a function of lateral ground displacement, illustrating that β_2 is not a constant, but rather it varies in the course of lateral spreading.

The equivalent linear p-δ relationship for the pile, defined by the degraded stiffness $\beta_2 k_2$, can be easily extended to a bilinear p-δ relationship by using the residual strength of liquefied soils in the definition of the ultimate lateral pressure from the liquefied soil on the pile. The empirical correlation between the undrained strength and SPT blow count proposed by Seed and Harder (1991) can be used for approximating the undrained strength in these calculations.

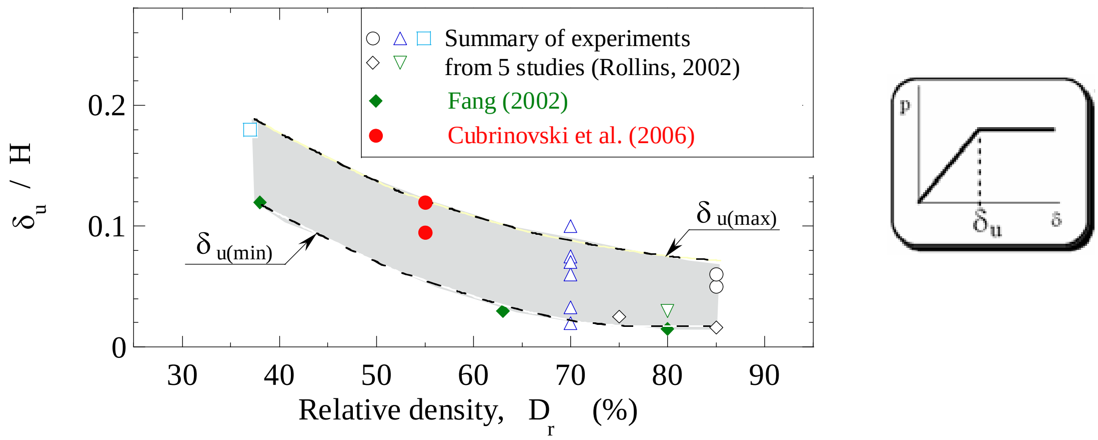

Figure 9. *Relative displacement required to fully mobilize the passive pressure as a function of the relative density of sand: summary of data from experimental studies*

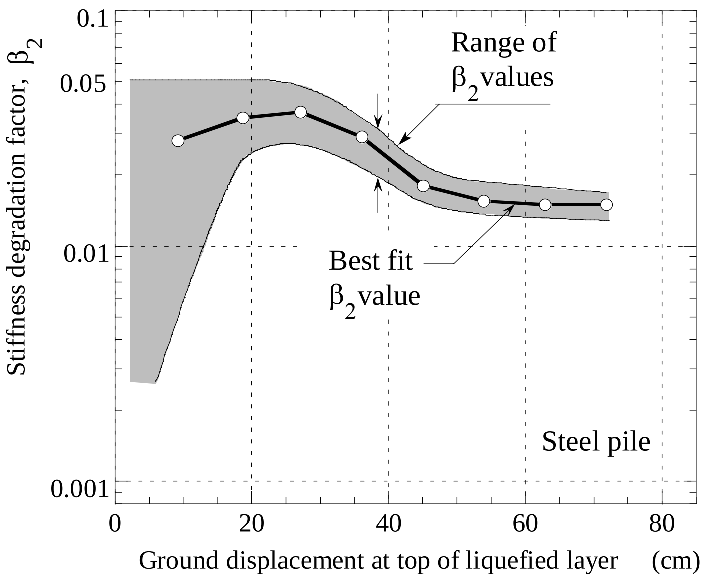

Figure 10. *Degradation of stiffness in the liquefied layer as a function of lateral ground displacement observed in full-size test on piles (Cubrinovski et al., 2006)*

Pile-Group Effects

Pile groups may generally affect the behaviour of piles in liquefying soils in two ways, first through the cross interaction among the piles within the group, and second, by influencing the key parameters controlling the pile response such as the stiffness of the liquefied soils, and the magnitude and spatial distribution of spreading displacements. Both effects are briefly discussed in the following.

Piles in a group are almost invariably rigidly connected at the pile head, and therefore, when subjected to lateral loads, all piles will share nearly identical horizontal displacements at the pile head. During lateral spreading of liquefied soils in a waterfront area, each of the piles will be subjected to a different lateral load from the surrounding soils, depending upon its particular location within the group and the spatial distribution of the spreading displacements (Figure 11). Consequently, both the interaction force at the pile head and the lateral soil pressure along the length of the pile will be different for each pile, thus leading to a development of distinct patterns of deformation and stresses along the length of individual piles in the group (Figure 12). This response feature, in which the piles share identical displacements at the pile head but have different deformations throughout the depth, is considered to be a significant feature of the deformational behaviour of pile groups subjected to lateral spreading. These pile-group effects can be easily captured by a simplified method of analysis using a single pile model (Cubrinovski and Ishihara, 2005).

The second influence of the pile-group regarding its effects on the magnitude and distribution of ground displacements, stiffness characteristics of spreading soils and ultimate soil pressure, is more difficult to quantify. Experimental data on these effects for piles in liquefiable soils is scarce and not conclusive. Figure 13 illustrates a clear tendency for reduction in the ultimate lateral soil pressure with increasing number of piles within the group, as compared to that of an individual pile. These data are for pile spacing of 2.5-3 diameters, and include both active and passive piles, though the trend is basically derived from active piles.

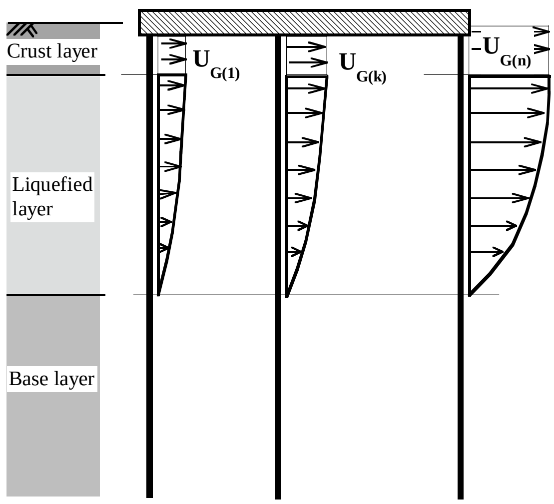

Figure 11: Piles in a group subjected to lateral ground displacements due to spreading

Further evidence for the pile-group effects on key parameters controlling the pile response in liquefying soils such as U_{G2}, β_2 and $p_{1\text{-}max}$ discussed herein is urgently needed.

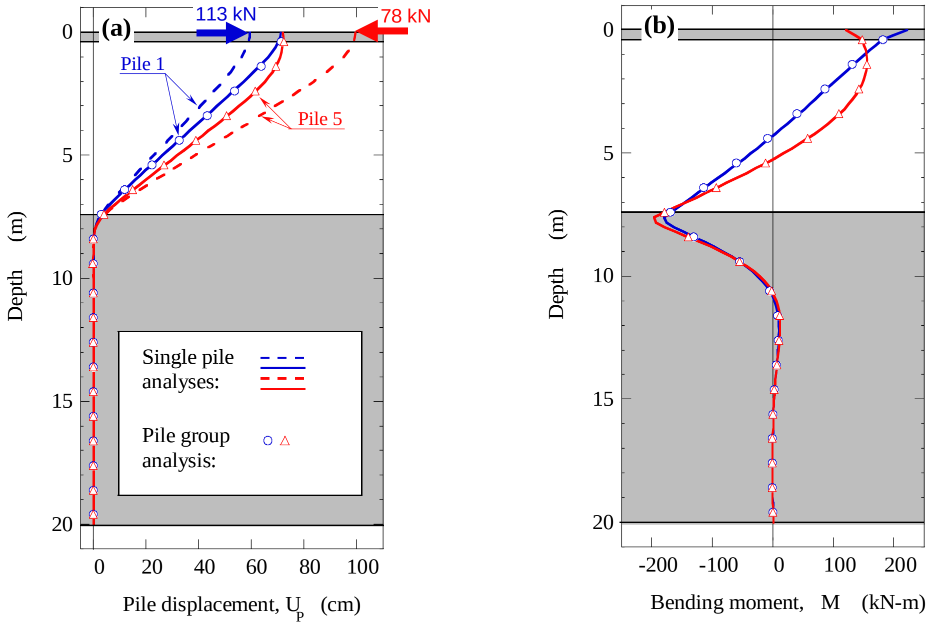

Figure 12: Illustration of cross-interaction effects on end piles subjected to different ground displacements (small ground displacement acting on Pile 1; large ground displacement acting on Pile 5); dashed lines indicate response of individual piles without cross –interaction effects; solid lines indicate response of individual piles including pile-group effects; (a) Pile displacements, and (b) Bending moments

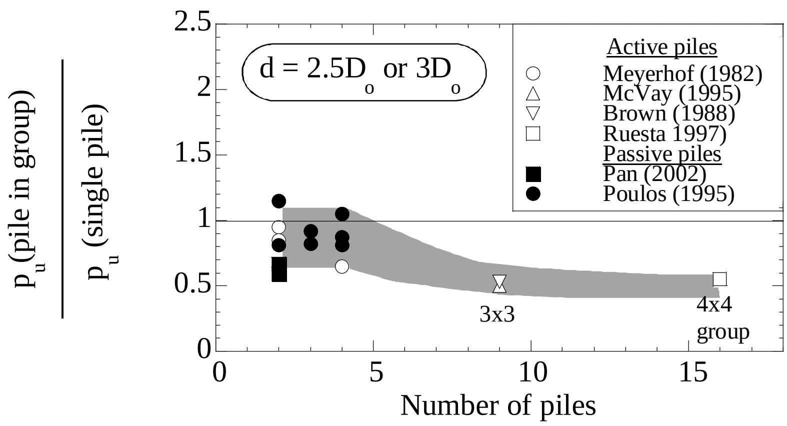

Figure 13: Reduction of lateral soil pressure due to pile-group effects

2.6 SUMMARIZED PROCEDURE FOR PSEUDO-STATIC ANALYSIS

For analysis of the cyclic phase of the pile response, key requirement is to concurrently consider the effects of ground displacements and inertial loads from the superstructure, and to properly consider the characteristics of liquefaction and subsequent ground displacements during the cyclic phase of loading. Cyclic ground displacements can be evaluated either by means of an effective stress analysis or by estimating the maximum cyclic shear strain in the liquefied soil based on empirical correlation. The maximum inertial load from the superstructure can be simply approximated as peak ground acceleration times supported vertical load by the pile.

The proposed practical procedure for preliminary assessment and design of piles subjected to lateral spreading can be summarized in the following steps:

1. A simplified three-layer model is developed for the soil deposit, where the liquefied layer is sandwiched between a non-liquefied crust layer at the ground surface and a non-liquefied base layer. The water table may be used in defining the thickness of the surface layer. Properties of the base layer within 4-6 pile diameters below the interface with the liquefied layer generally control the p-δ relationship of the base layer. A single pile with a nonlinear moment-curvature relationship is adopted in this model.
2. The magnitude of lateral spreading displacement can be estimated using empirical correlations for ground surface displacements of lateral spreads. In view of the uncertainties involved in the assessment of these displacements, a range of values needs to be considered. It is practical to assume a cosine distribution for the ground displacement within the liquefied soil and that the surface layer will move together with the top of the liquefied soil.
3. Initial stiffness in all p-δ relationships can be defined based on empirical correlations between the subgrade reaction coefficient and SPT blow count or elastic moduli. This stiffness should then be degraded in order to account for the effects of nonlinearity and large relative displacements that are required to fully mobilize the lateral soil pressure.
4. Stiffness degradation of liquefied soils is generally in the range between 1/50 and 1/10 for cyclic liquefaction and 1/1000 to 1/50 for lateral spreading.
5. Ultimate lateral pressure from the crust layer can be approximated as being 4.5 times the Rankine passive pressure. Empirical charts for the residual strength of liquefied soils can be used for the ultimate lateral pressure from the liquefied soil on the pile.
6. A static analysis in which the pile is subjected to ground displacements defined in step 2, and adopted stiffness degradation in step 4, is performed and pile displacements and bending moments are obtained. The analysis should be repeated while parametrically varying the magnitude of applied ground displacement and stiffness degradation in the liquefied soil.
7. Pile group effects should be eventually considered including cross interaction among the piles within the group through the pile-cap-pile system, and effects on key parameters controlling the pile response such as the stiffness and strength of liquefied soils, and the magnitude and spatial distribution of

spreading displacements. The latter effects may potentially reduce the severity of the ground movement influence.

2.7 CONCLUSIONS

Lateral ground displacements of liquefied soils can be quite large during the intense shaking or cyclic phase of loading and especially during the post-liquefaction lateral spreading phase. Since the properties of liquefied soils and loads on piles can be remarkably different during the cyclic phase and subsequent spreading phase, it is necessary to separately consider these two phases in the simplified analysis of piles. When evaluating the pile response during the cyclic phase it is important to consider a relevant combination of kinematic loads due to cyclic ground displacements and inertial loads from the superstructure. In the case of lateral spreading, the uncertainties associated with the spreading of liquefied soils, and in particular, the magnitude and the spatial distribution of spreading displacements, as well as stiffness and strength degradation of liquefied soils need to be carefully considered. The lateral load from a non-liquefiable crust layer at the ground surface may often be the critical load for the integrity of piles subjected to lateral spreading, and therefore, special attention needs to be given to the modelling of the surface layer and its effects on the pile response. Cross-interaction effects may be significant for pile foundations near the waterfront area, where individual piles within the group are subjected to variable ground displacements. Effects of group interaction on key parameters controlling the pile response need to be considered in perhaps reducing the severity of the ground movement effects. Based on these premises, a practical procedure for preliminary assessment and design of piles subjected to lateral spreading has been proposed.

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CHAPTER 3: ANALYSIS OF PILES IN LIQUEFYING SOILS BY THE PSEUDO-STATIC APPROACH - SENSITIVITY STUDY

Cubrinovski, M., Haskell, J. and Bradley B. (2010). Advances in Geotechnical Earthquake Engineering, Chapter 20: Analysis of piles in liquefying soils by the pseudo-static approach, Springer (in print).

Abstract

The pseudo-static method of analysis is a simplified design-oriented approach for analysis of seismic problems based on routine computations and conventional engineering models. The application of the method to analysis of piles in liquefying soils is burdened by significant uncertainties associated with soil liquefaction, soil-pile interaction in liquefying soils and the need to reduce a very complex dynamic problem to a simple equivalent static analogy. Hence, despite its simplicity, the application of the pseudo-static analysis is not straightforward and requires careful consideration of the uncertainties in the analysis. This paper addresses some of the key issues that arise in the application of the pseudo-static analysis to piles in liquefying soils, and makes progress towards the development of a clear modelling (analysis) strategy that permits a consistent and reliable use of the simplified pseudo-static analysis.

A comprehensive parametric study has been conducted in which a wide range of soil-pile systems, loading conditions and values for model parameters were considered. In the analyses, the pile response during the strong ground shaking (cyclic phase) and post-liquefaction lateral spreading was considered by two separate pseudo-static analysis approaches. In this paper, results from the analyses are used to examine and quantify the sensitivity of the pile response to various model parameters, and to establish a fundamental link between the sensitivity of the pile response and the mechanism of soil-pile interaction. On this basis, some general principles for conducting pseudo-static analysis of piles in liquefying soils could be established irrespective of the specific properties of the soil-pile system and loading conditions.

Keywords: *Liquefaction, lateral spreading, pile, pseudo-static analysis, uncertainties*

3.1 INTRODUCTION

Methods for assessment of the seismic performance of pile foundations in liquefying soils have evolved significantly over the past two decades. The 1995 Kobe earthquake, in particular, contributed to an improved understanding of the complex behaviour of piles in liquefying soils through evidence from well-documented case histories (JGS, 1998) and benchmark experimental studies that this event instigated (e.g. Cubrinovski et al., 2006; Tokimatsu and Suzuki, 2009). On the analytical front, significant progress has been made across a very broad and diverse group of analysis methods ranging from simple design-oriented approaches to the most advanced (and complex) numerical procedures for dynamic analysis (O'Rourke et al., 1994; Tokimatsu and Asaka, 1998; Yasuda and Berrill, 2000; Finn and Thavaraj, 2001; Cubrinovski et al., 2008).

Nominally, all these analysis methods have the same objective, to assess the seismic performance of the pile foundation and evaluate ground displacements, pile deformations and damage to piles. However, a close inspection of different methods reveals that they each focus on different aspects of the problem and provide a distinct contribution in the assessment of seismic performance (Cubrinovski and Bradley, 2009). For example, Table 1 summarizes key features of three representative methods of analysis: (1) Pseudo-static analysis using a conventional beam-spring model (a simple design-oriented approach), (2) Seismic effective stress analysis (an advanced dynamic analysis incorporating effects of excess pore pressures and dynamic soil-pile-structure interaction), and (3) Probabilistic assessment within the Performance-Based Earthquake Engineering (PBEE) framework (a method rigorously quantifying the uncertainties and seismic risk). As outlined in Table 1, each of these methods of analysis provides significant and different contribution in the assessment, and importantly, all methods have some shortcomings. In essence, these analysis methods are complementary in nature and it is envisioned that they will be all used in parallel in the future, hence they all require further development and improvement.

The pseudo-static method of analysis is a practical engineering approach based on routine computations and the use of relatively simple models. Application of this analysis method does not require excessive computational resources nor specialist knowledge, and it is thus a widely-adopted approach in current practice and seismic design codes. The application of the method to the analysis of piles in liquefying soils is not straightforward however, but rather is burdened by significant uncertainties associated with soil liquefaction during earthquakes, soil-pile interaction in liquefying soils and the need to reduce a very complex dynamic problem to a simple equivalent static analogy. Questions posed to the user are 'what stiffness and strength to adopt for the liquefied soil', 'how to combine oscillatory kinematic and inertial loads in a static analysis' and 'what is the sensitivity of the pile response to a certain model parameter', among others. This paper highlights the issues around the implementation of the pseudo-static method of analysis for piles in liquefying soils and the effects of uncertainties in the analysis. It summarizes some of the key findings from a systematic analytical study and points towards methodology for a consistent and reliable use of the simplified pseudo-static analysis.

Table 1. Methods for assessment of seismic performance of soil-structure systems: key features and contributions

Method of assessment	Key features	Specific contributions in the assessment	Shortcomings
Pseudo-static analysis	• Simple to use • Conventional data and engineering concepts	• Evaluates the response and damage to the pile (**parametric evaluation is needed)** • Enhances foundation design through better understanding of soil-pile interaction mechanism	• Gross approximation of dynamic loads and behaviour • Aims at maximum response only
Seismic effective stress analysis	• Realistic simulation of ground response and seismic soil-foundation-structure interaction • Complex numerical procedure	• Detailed assessment of seismic response of pile foundations including effects of liquefaction and SSI • Considers inelastic behaviour of the entire soil-foundation-structure system • Enhances communication of design between geotechnical and structural engineers	• Ignores uncertainties in the ground motion and numerical model • High demands on the user
Probabilistic PBEE framework	• Considers 'all' earthquake scenarios • Quantifies seismic risk	• Addresses uncertainties associated with ground motion and numerical model on a site specific basis • Provides engineering measures (response and damage) and economic measures (losses) of performance • Enhances communication of design and seismic risk outside profession	• Ignores details of the seismic response

3.2 SOIL-PILE INTERACTION IN LIQUEFYING SOILS

Cyclic phase

Soil-pile interaction in liquefying soils involves significant changes in soil stiffness, strength and lateral loads on the pile over a very short period of time during and immediately after the strong ground shaking. As illustrated in Figure 1a, the excess pore pressure may reach the level of the effective overburden stress in only a few seconds of strong shaking, and this is practically the time over which the soil stiffness reduces from its initial value to nearly zero. The intense reduction in stiffness and strength of the liquefying soil is accompanied by large lateral ground displacements, as illustrated with the solid line in Figure 1b. Hence, during this phase of strong ground shaking (high accelerations) and development of liquefaction, the piles are subjected to significant kinematic loads due to lateral ground movement along with inertial loads from vibration of the superstructure (Figure 1c). Both these loads are oscillatory in nature with magnitudes and spatial distribution dependent on a number of factors, including ground motion characteristics, soil density, the presence of non-liquefied crust at the ground surface, predominant periods of the ground and superstructure, and the relative stiffness of the foundation soil and the pile, among others.

Lateral spreading phase

In sloping ground or backfills behind retaining structures, liquefaction results in unilateral ground displacements or lateral spreading, as illustrated schematically with the dashed line in Figure 1b. Lateral spreading typically results in large permanent ground displacements of up to several meters in the down-slope direction or towards waterways.

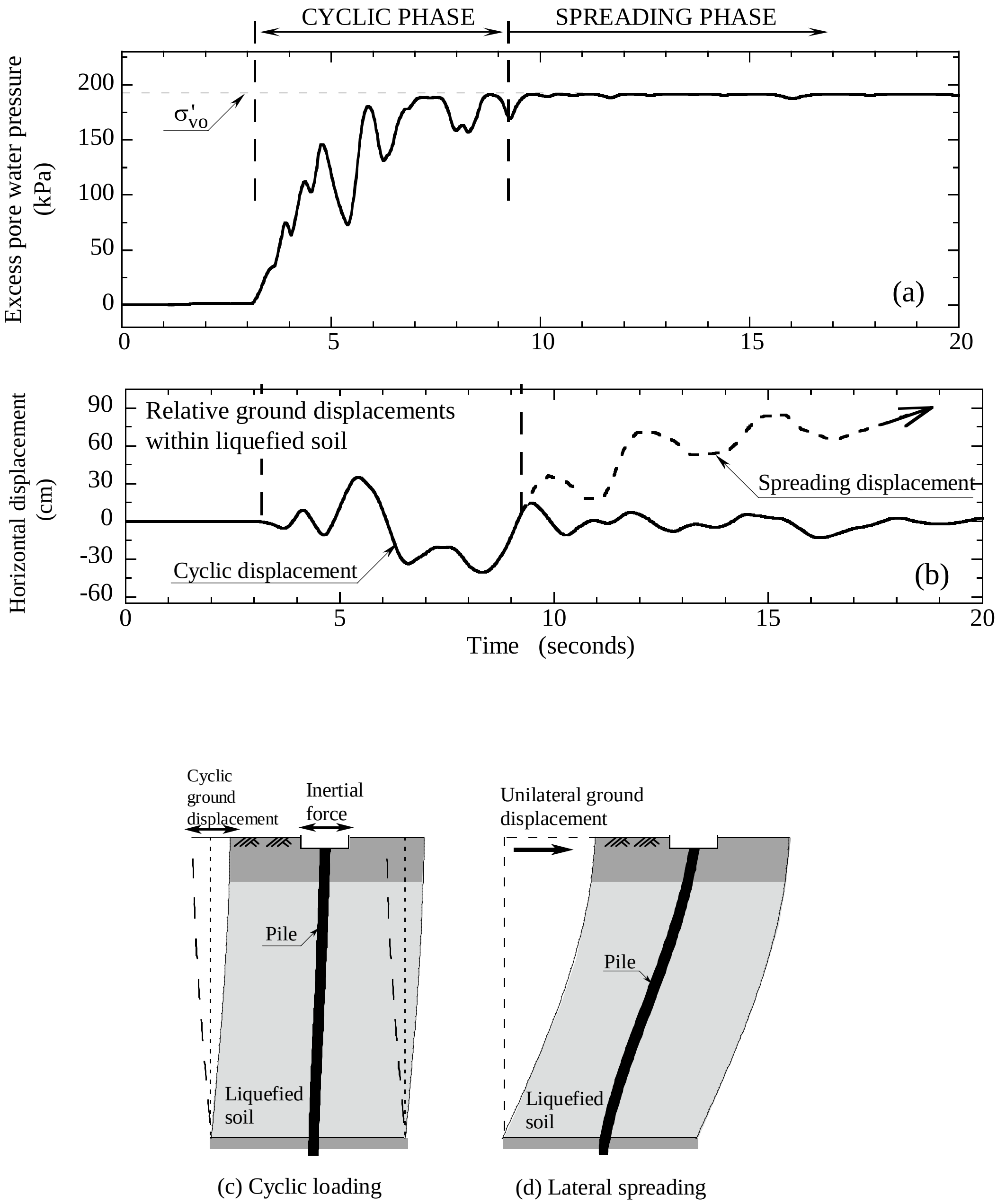


Figure 1: ***Illustration of ground response and soil-pile interaction in liquefying soils: (a) Excess pore water pressure; (b) Lateral ground displacement; (c) Loads on pile during strong ground shaking (cyclic phase), and (d) post-liquefaction lateral spreading***

There are many possible scenarios for the spatial and temporal distribution of lateral spreading displacements, depending on the stress-strain characteristics of soils, gravity-induced driving shear stresses and ground motion features. In general, lateral spreading may be initiated during the intense pore pressure build up and onset of liquefaction, however spreading displacements may continue well after the development of complete liquefaction and after the end of the strong shaking. One may argue that spreading is a post-liquefaction phenomenon or at least that a significant portion of the spreading displacements occurs after the foundation soils have liquefied. The spreading displacements may be one order of magnitude greater than the cyclic ground displacements, while the inertial loads during spreading are comparatively small. This is reflected in the schematic plot in Figure 1d, where the inertial loads on the pile have been ignored. Thus, both the characteristics of the foundation soil and the lateral loads on piles are very different between the cyclic phase and the subsequent lateral spreading phase, and therefore, these two phases should be considered separately in the simplified pseudo-static analysis of piles.

3.3 PSEUDO-STATIC APPROACH FOR SIMPLIFIED ANALYSIS

As a practical design-oriented approach, the pseudo-static analysis needs to be relatively simple, based on conventional geotechnical data and engineering concepts. In order to satisfy the objectives in the seismic performance assessment however, the pseudo-static analysis of piles also should: (a) capture the relevant deformational mechanism for piles in liquefying soils, (b) permit estimation of inelastic deformation and damage to piles, and (c) address the uncertainties associated with seismic behaviour of piles in liquefying soils. The adopted model in this study was developed based on this reasoning.

Although the pseudo-static analysis could in principle be applied to a pile group, it is often applied to a single-pile model. This approach was adopted in the analyses presented herein, as pile group effects introduce further complexities to the problem, beyond the scope of this study. For the same reason, effects of axial loads and geometric nonlinearity were ignored in this study. A typical beam-spring model representing the soil-pile system in the simplified pseudo-static analysis is shown in Figure 2 (Cubrinovski and Ishihara, 2004; Cubrinovski et al., 2009a). The model can easily incorporate a stratified soil profile (multi-layer deposit) with liquefied layers of different thicknesses sandwiched between a crust of non-liquefiable soil at the ground surface and an underlying non-liquefiable base layer. Given that a key requirement of the analysis is to estimate the inelastic deformation and damage to the pile, the proposed model incorporates simple non-linear load-deformation relationships for the soil and the pile. The soil is represented by bilinear springs, the stiffness and strength of which can be degraded to account for effects of nonlinear behaviour and liquefaction. The pile is modelled using a series of beam elements with a tri-linear moment-curvature relationship. Parameters of the model are summarized in Figure 2b for a typical three-layer configuration in which a liquefied layer is sandwiched between surface layer and base layer of non-liquefiable soils.

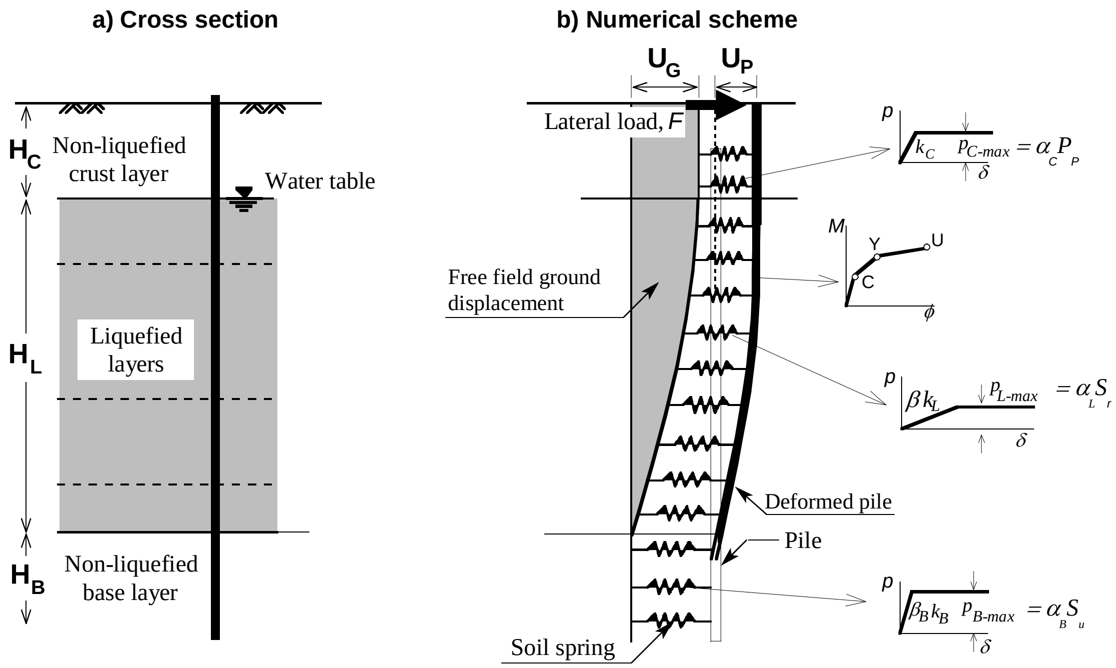


Figure 2: Beam-spring model for pseudo-static analysis of piles in liquefying soils: model parameters and characterization of nonlinear behaviour (Cubrinovski and Ishihara, 2004; Cubrinovski et al., 2009a)

In the model, two equivalent static loads are applied to the pile: a lateral force at the pile head (F), representing the inertial load on the pile due to vibration of the superstructure, and a horizontal ground displacement (U_G) applied at the free end of the soil springs (for the liquefied layer and overlying crust), representing the kinematic load on the pile due to lateral ground movement (cyclic or spreading) in the free field. As indicated in Figure 2, it has been assumed that the non-liquefied crust at the ground surface is carried along with the underlying liquefied soil and that it undergoes the same ground displacement as the top of the liquefied layer, U_G.

3.4 MODEL PARAMETERS

The aim of the pseudo-static analysis is to estimate the maximum response of the pile induced by an earthquake, under the assumption that dynamic loads can be idealized as static actions. The key question in its implementation is thus how to select appropriate values for the soil stiffness, strength and lateral loads on the pile in the equivalent static analysis. In other words, what are the appropriate values for β_L, $p_{L\text{-}max}$, U_G and F in the model shown in Figure 2? The following discussion demonstrates that this choice is not straightforward and that each of these parameters may vary within a wide range of values.

Crust layer

The lateral load from a crust of non-liquefied soil at the ground surface may often be the critical load for the integrity of the pile because of its potentially large magnitude and unfavourable position as a "top-heavy" load, acting above the unsupported portion of the pile embedded in liquefied soils.

The ultimate soil pressure from the surface layer per unit width of the pile can be estimated, for example, using a simplified expression such as, $p_{C\text{-}max} = \alpha_C\, p_p$, where $p_p(z)$ is the Rankine passive pressure while α_C is a scaling factor to account for the difference in the lateral pressure between a single pile and an equivalent wall. Figure 3 summarizes values for α_C derived from experimental studies on piles which include benchmark lateral spreading experiments on full-size piles (Cubrinovski et al., 2006). In those experiments, the maximum lateral pressure on the single pile was found to be about 4.5 times the Rankine passive pressure. The very large values for the parameter α_C shown in Figure 3 clearly indicate that excessive lateral loads can be applied from the crust layer to the pile. Notable also is a relatively significant variation in the values of α_C in the range between 3 and 5.

Liquefied layer

The stiffness degradation factor β_L, which specifies the reduction of stiffness due to liquefaction and nonlinear stress-strain behaviour ($\beta_L k_L$), is affected by a number of factors including the excess pore pressure level, magnitude of ground displacements, density of sand, and drainage conditions. Typically, β_L takes values in the range between 1/50 and 1/10 for cyclic liquefaction (Tokimatsu and Asaka, 1998) and between 1/1000 and 1/50 in the case of lateral spreading (Yasuda and Berrill, 2000; O'Rourke et al. 1994; Cubrinovski et al., 2006). In general, the value of β_L should be related to the soil properties and anticipated ground deformation. For example, lower values of β_L are expected for very loose soils because such soils are commonly associated with high and sustained excess pore water pressures and large ground deformation. While this sort of qualitative evaluation of β_L should be considered, the quantification of the value for this parameter is very difficult and subjective because of the inherent uncertainties associated with properties of liquefying soils.

The residual strength of liquefied soils S_r could be used in the evaluation of the ultimate pressure from the liquefied soil on the pile, e.g., $p_{L\text{-}max} = \alpha_L S_r$. Here, the residual strength S_r can be estimated using an empirical correlation between the residual strength of liquefied soils and SPT blow count, such as that proposed by Seed and Harder (1991) or Olson and Stark (2002). Note that the former correlation assumes a constant residual strength for a given blow count while the latter one uses a normalized residual strength of the form (S_r/σ'_{vo}) and hence implicitly assumes that S_r increases with depth for a given SPT blow count. Effects of strength normalization on the pile response are beyond the scope of this paper, but detailed analysis of these effects may be found in Cubrinovski et al., 2009b).

The shaded area in Figure 4 shows the correlation between S_r and the normalized SPT blow count for clean sand $(N_1)_{60cs}$ proposed by Seed and Harder (1991). A large scatter exists in this correlation indicating significant uncertainty in the value of S_r for a given $(N_1)_{60cs}$ blow count. For example, for $(N_1)_{60cs}$ = 10, the value of S_r can be anywhere between 5 kPa (lower bound value) and 25 kPa (upper bound value). In addition, the multiplier α_L ($p_{L\text{-}max} = \alpha_L\, S_r$) is also unknown and subject to significant uncertainties. Note that α_L is different from the corresponding parameter α_C for the crust layer previously discussed, because the interaction and mobilization of pressure from surrounding soils on the pile is different for liquefied and non-liquefied soils.

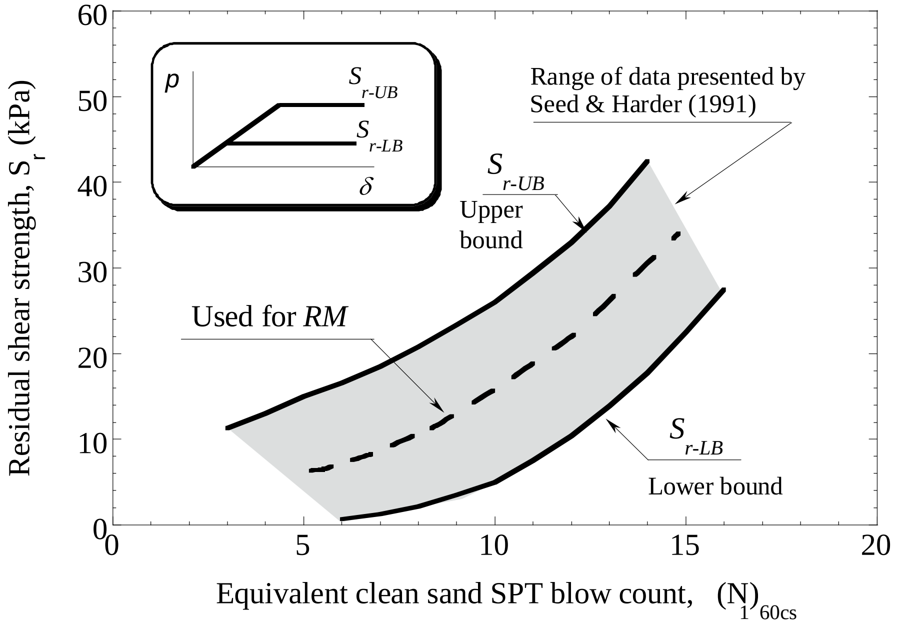


Figure 4: Residual shear strength of liquefied soils (after Seed and Harder, 1991)

Other parameters

As indicated in Figure 2, there are a number of other parameters of the soil-pile model that may influence the response of the soil-pile system, such as the parameters of the base layer or the initial (non-degraded) stiffness of soil springs. The latter is represented in the model by the subgrade reaction coefficient (k_i). A flow chart showing the various soil properties and relationships that are used to determine the model parameters for the soil springs is shown in Figure 5. Note that both stiffness and strength properties in the model are derived based on the SPT blow count using conventional approaches (Rankine passive pressure theory, subgrade reaction coefficient) and empirical expressions, hence the selection of an appropriate representative blow count could be critical in the evaluation of the pile response.

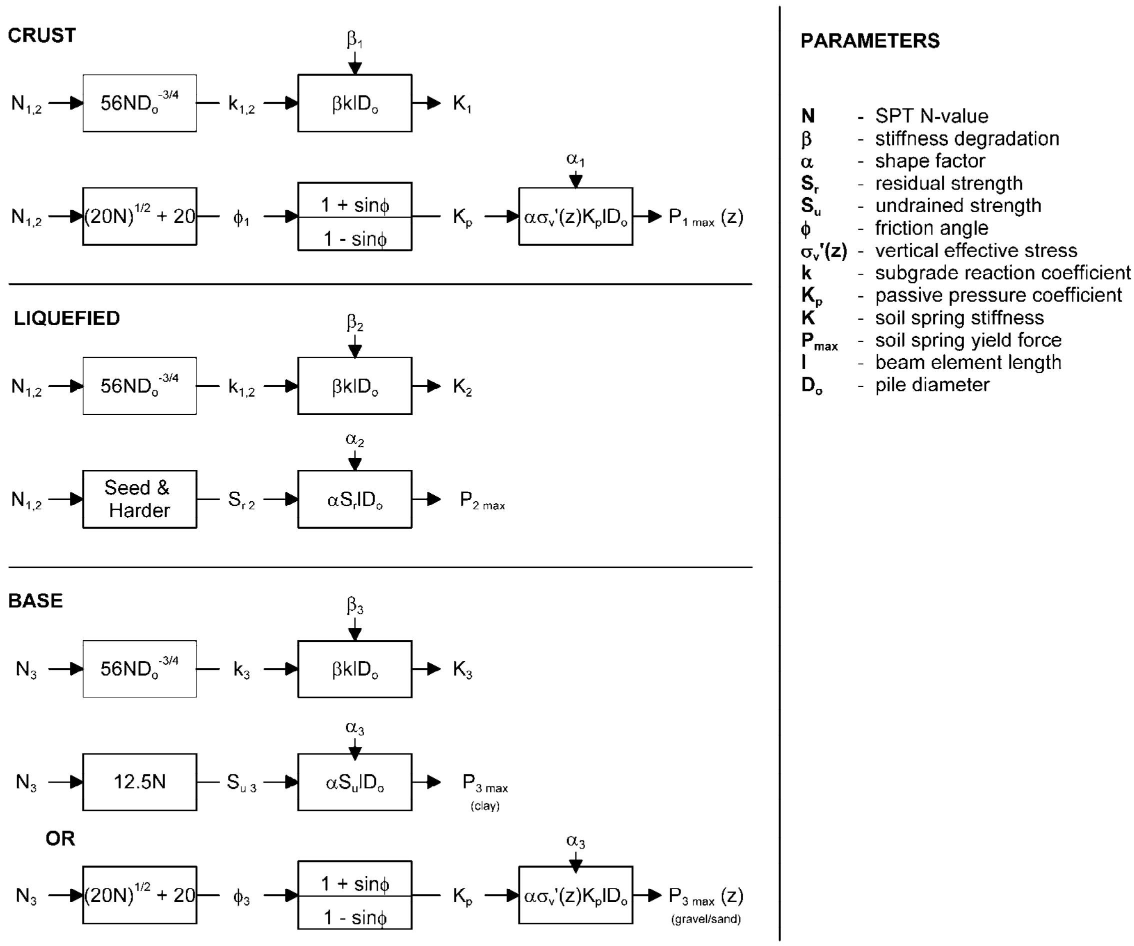


Figure 5: ***Flow chart illustrating the determination of soil spring parameters in the computational model (Note: subscripts 1, 2 and 3 denote crust, liquefied and base layer respectively)***

Equivalent static loads

The selection of appropriate equivalent static loads is probably the most difficult task in the pseudo-static analysis. This is because both input loads in the pseudo-static analysis (U_G and F) are, in effect, estimates of the seismic response of the free field ground and superstructure respectively. Simplified methods for the evaluation of lateral displacements of liquefied and laterally spreading soils (e.g., Tokimatsu and Asaka, 1998; Youd et al., 2002) and guidance how to combine kinematic and inertial loads on piles in liquefying soils (Boulanger et al., 2007; Tokimatsu and Suzuki, 2009) are now available, however they all imply significant uncertainties in these loads.

Implementation of PSA

The significant uncertainties associated with the model parameters in the pseudo-static analysis need to be considered in the assessment of the pile response. One of the first questions that needs to be answered is not 'what is the most appropriate value for a given model parameter', but rather 'how big is the effect of variation (uncertainty) in a given model parameter on the predicted pile response'. This will allow the user to focus their attention on critical uncertainties in the analysis and develop a suitable strategy for a robust implementation of the simplified pseudo-static analysis.

3.5 CONCEPT OF THE SENSITIVITY STUDY

To investigate the sensitivity of the pile response to various model parameters and hence identify critical uncertainties in the pseudo-static analysis, a comprehensive parametric study has been conducted in which a wide range of soil-pile systems, loading conditions and values for model parameters were considered (Haskell, 2009). The objectives of the analyses were to comparatively examine and quantify the sensitivity of the pile response to various model parameters, and to establish a fundamental link between the sensitivity of the pile response and the mechanism of soil-pile interaction. In this way, general principles for conducting pseudo-static analysis of piles in liquefying soils could be established irrespective of the specific properties of the soil-pile system or loading conditions.

Four different soil profiles, shown in Figure 6, were adopted for the parametric study. Each profile essentially consists of two layers, a 10m thick liquefiable sand layer overlying a 10m thick non-liquefiable base layer. A loose sand with an SPT blow count of N = 5 was adopted for the liquefiable soil in profiles *P1* and *P2*, whereas a medium-dense sand with N = 15 was used for profiles *P3* and *P4*. Similarly, a soft clay base layer was adopted for profiles *P1* and *P3*, whereas a dense sand base layer was used in profiles *P2* and *P4*. The soil above the water table was assumed to act as a non-liquefiable crust at the ground surface, and five different scenarios were adopted for the location of the water table between z = 0m and 2m depth defining a crust with thickness of H_C = 0., 0.5, 1.0, 1.5 and 2.0m respectively. For all cases, three different piles were considered, with diameters of 400mm, 800mm and 1200mm respectively, and tri-linear M-ϕ relationships as summarized in Table T2.

As discussed earlier, the characteristics of liquefying soils and demands on piles are significantly different during the cyclic phase and lateral spreading phase of the response. For this reason and for clarity of the argument, three separate series of analyses were conducted, covering the following loading conditions: (1) Lateral Spreading Scenario, (2) Hypothetical Cyclic Scenario (without inertial force demand), and (3) Cyclic Scenario. Even though the actual sequence of the phenomena has been practically reversed and the second scenario is purely hypothetical in nature (based on an unrealistic assumption), the results of the analyses will be presented in the abovementioned order because it will help the clarity of the presentation and allow for gradual introduction of complexities in the interpretation of results.

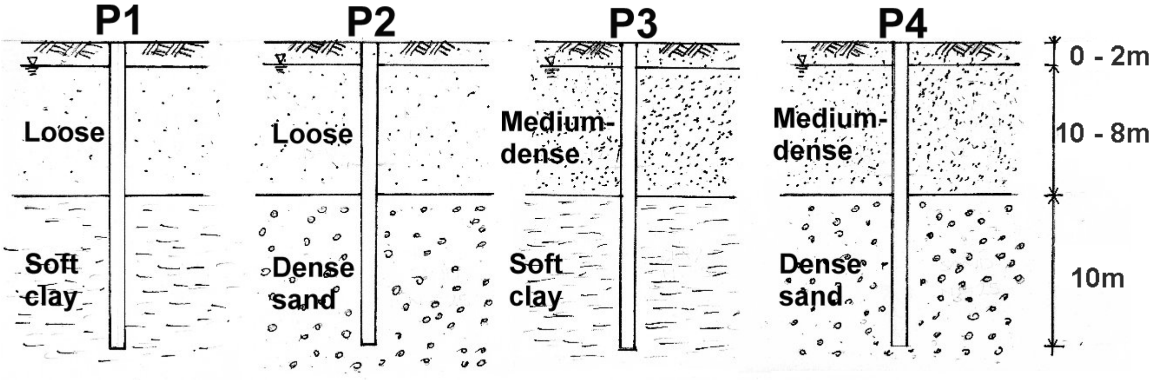


Figure 6. Soil profiles adopted in the parametric studies

Table 2. Tri-linear moment-curvature relationships of piles used in the parametric studies

	Units	*S-Pile*	*M-Pile*	*F-Pile*
D	[mm]	1200	800	400
M_C	[kN-m]	959	650	80.4
M_Y	[kN-m]	1970	1240	126
M_U	[kN-m]	2470	1420	133
ϕ_C	[m^{-1}]	0.00041	0.00109	0.00190
ϕ_Y	[m^{-1}]	0.00251	0.00609	0.01160
ϕ_U	[m^{-1}]	0.01040	0.01169	0.02450

3.6 LATERAL SPREADING PSEUDO-STATIC ANALYSES

As described earlier, there are significant uncertainties in the pseudo-static analysis (PSA) around the selection of appropriate values for model parameters. Thus, for a given soil-pile system and loading conditions, a relatively wide range of values could reasonably be used in the PSA for each parameter. Table T3 summarizes the ranges of values for all parameters of the pseudo-static model, for the four adopted soil profiles. The ranges of values for different parameters have been defined on the basis of well-documented case histories, evidence from experimental studies, and numerical analyses. Discussion on this can be found in Cubrinovski et al., 2006, Cubrinovski and Ishihara (2007) and Haskell (2009). Three values are listed in the table for each model parameter: a lower bound or minimum acceptable value (LB), a reference or ‘mid-range’ value, and an upper bound or maximum acceptable value (UB).

For each soil-pile system (e.g. *S-P1,* S-Pile in soil profile P1), an analysis was first conducted using the reference values for all model parameters (*RM* = Reference Model), thus establishing a reference pile response. Next, sensitivity analyses were carried out considering one model parameter at a time. For example, when examining the sensitivity of the pile response to β_L, an analysis was conducted in which the lower bound value of $\beta_L = \beta_{L\text{-}LB} = 0.001$ was used while all other parameters were set at their reference values. Another analysis was then conducted using instead the upper bound value for $\beta_L = \beta_{L-UB} = 0.02$, with all other parameters again being set at their reference values. From these two analyses, the sensitivity of the pile response to the β_L-value could be examined. The same procedure was followed and systematically applied to establish the sensitivity of the pile response to each of the model parameters.

In the lateral spreading analyses, seven different loading conditions (magnitudes of lateral ground displacements) were applied, i.e. U_G = 0.1, 0.25, 0.50, 0.75, 1.0, 1.5 and 2.0 m respectively. Thus, for each case (soil-pile system and model parameter), the sensitivity was evaluated for seven different load levels. This approach was taken because the sensitivity of the response was expected to depend on the induced mechanism of soil-pile interaction which, in turn, depends on the induced pile response (displacement) and hence applied load to the pile (ground displacement).

Table 3. Parameter variations considered in the sensitivity analyses

		Profile – P1			Profile – P2			Profile – P3			Profile – P4		
	Units	LB	Ref	UB	LB	Ref	UB	LB	Ref	UB	LB	Ref	UB
β_C	-	0.3	1	1	0.3	1	1	0.3	1	1	0.3	1	1
α_C	-	3	4.5	5	3	4.5	5	3	4.5	5	3	4.5	5
$\Phi_{C,L}$	°	27	30	33	27	30	33	34	37	40	34	37	40
$k_{C,L}$	MNm^{-3}	$168D^{-3/4}$	$280D^{-3/4}$	$392D^{-3/4}$	$168D^{-3/4}$	$280D^{-3/4}$	$392D^{-3/4}$	$504D^{-3/4}$	$840D^{-3/4}$	$1176D^{-3/4}$	$504D^{-3/4}$	$840D^{-3/4}$	$1176D^{-3/4}$
$N_{C,L}$	-	2	5	8	2	5	8	11	15	19	11	15	19
β_L	-	0.001	0.01	0.02	0.001	0.01	0.02	0.001	0.01	0.02	0.001	0.01	0.02
α_L	-	1	3	6	1	3	6	1	3	6	1	3	6
S_r	kPa	1	7	15	1	7	15	25	34	44	25	34	44
β_B	-	0.3	1	1	0.3	1	1	0.3	1	1	0.3	1	1
k_B	MNm^{-3}	$168D^{-3/4}$	$280D^{-3/4}$	$392D^{-3/4}$	$840D^{-3/4}$	$1400D^{-3/4}$	$1960D^{-3/4}$	$168D^{-3/4}$	$280D^{-3/4}$	$392D^{-3/4}$	$840D^{-3/4}$	$1400D^{-3/4}$	$1960D^{-3/4}$
α_B	-	5	9	9	3	4.5	5	5	9	9	3	4.5	5
Su_B	kPa	25	33	42	432	496	573	25	33	42	432	496	573
N_B	-	2	5	8	20	25	30	2	5	8	20	25	30

Parametric sensitivities

Results from the sensitivity analyses for *S-P1* (*S-Pile* in soil profile *P1*) with a crust of H_C = 1.5m are presented in Figure 7 in the form of 'tornado charts'. Here, the pile response is presented using the peak curvature (left-hand side plots in Figure 7) and pile head displacement (right-hand side plots in Figure 7) computed in PSA. The former illustrates the level of damage to the pile while the latter provides information on the relative displacement between the pile and the soil, and thus indicates the respective soil-pile interaction mechanism. For example, the (red) bold line in Figure 7j indicates that the pile displacement computed using the reference model parameters (RM) was U_P = 0.21m in the analysis in which a lateral ground displacement of U_G = 1.0m was applied. Hence, the relative displacement between the soil and the pile was $\delta = U_G - U_P$ = 0.79m, indicating that the soil springs in the liquefied and crust layers yielded. This is illustrated schematically with the sketches in Figure 7, where yielding in the soil is indicated (in red) showing the depth along which the ultimate soil pressure was mobilized and applied to the pile.

Each bar in the tornado charts indicates the sensitivity of the pile response to the variation of a specific parameter. The left end of the bar shows the response computed when a lower bound value was used for a parameter of the liquefied or crust layer, while the right end of the bar was obtained in a respective analysis with an upper-bound value for the parameter. Note that the reverse is true for the base layer where the maximum response (right end of the bar) was obtained when lower bound values were used for the base layer. This simply reflects the fact that the liquefied layer and the crust provide driving forces to the pile deformation whereas the base layer provides a resisting force.

Note that in the tornado charts, model parameters for the surface layer, liquefied layer and base layer are ordered respectively from top to bottom for clearer illustration of the sensitivity of the pile response with regard to a particular soil layer. There are several key findings from the results presented in Figure 7:

- The bars for parameters of the base layer (β_B, k_B, α_B and $S_{u\text{-}B}$) are relatively small in size showing that the pile response is less sensitive to the variation in the parameters of the base layer as compared to those of the crust or liquefied layer. Hence, when using PSA, parametric studies should focus on the parameters of the liquefied soil and non-liquefied layer at the ground surface.
- For small relative displacements $|U_G - U_P|$, the pile response is most sensitive to parameters affecting the stiffness of the soil (β_L). As the relative displacement increases and yielding in the soil occurs, the response becomes more sensitive to the strength of the soil (parameters α_L and S_r). This transition from stiffness to strength controlled pile response is schematically illustrated in Figure 8 where the change in the size of the bar in the tornado chart is clearly related to the mobilized load (deformation) of the soil spring relative to the yield level.
- The response is sensitive to the SPT blow count reflecting its concurrent use for computing both the soil stiffness and the soil strength parameters.

Note that for this soil-pile system (*S-P1*), the displacement of the pile head was always less than the ground displacement, resulting in so-called stiff-pile behaviour in which the pile resists the lateral spreading displacements of the liquefied soil and non-liquefied crust.

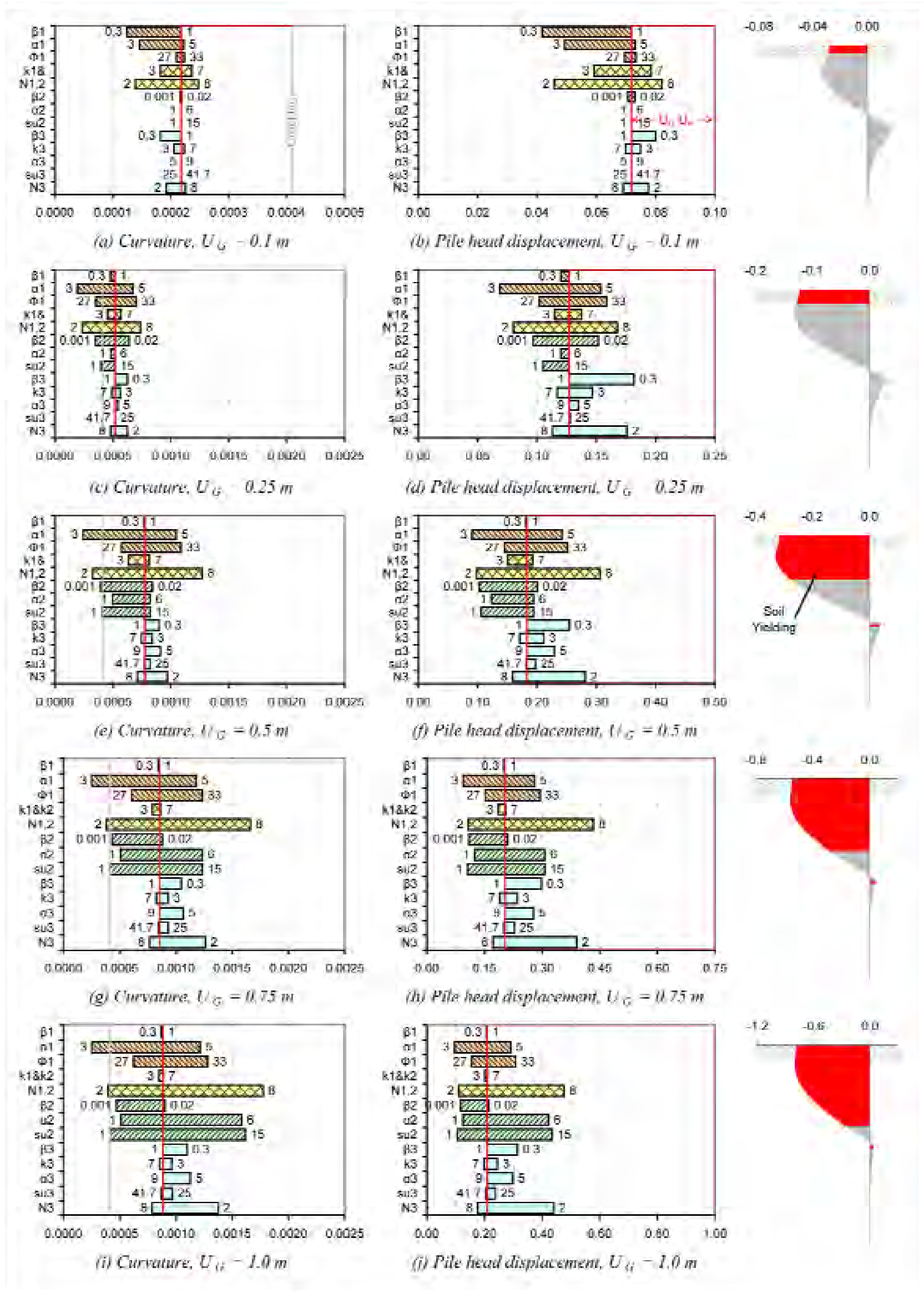


Figure 7. 'Tornado charts' illustrating sensitivity of pile response (peak curvature and pile head displacement) for selected ground displacements ranging from 0.1m to 1.0m (lateral spreading PSA); (Note: subscripts 1, 2 and 3 denote crust, liquefied and base layer respectively)

Parameters with relatively small influence on pile response

As discussed earlier, the pile response is relatively insensitive to the properties of the base layer. The stiffness degradation of the crust soil, β_C, is also not considered a critical uncertainty as it only affects the pile response at very small (and not so relevant) ground displacements. Similarly, the uncertainties associated with the subgrade reaction coefficients (k_i) arising from the empirical relationship based on SPT blow count generally have negligible effects on the pile response.

Critical uncertainties

It has already been noted that changes in the SPT blow count of all soil layers have a significant influence on the pile response. This highlights the need for careful selection of a 'representative blow count' for each soil layer and the need for assessment of the effects of uncertainties in the SPT blow count on the pile response. Unlike other model parameters, the SPT blow count simultaneously affects multiple soil properties (stiffness and strength of soils, as illustrated in Figure 5), and therefore, it shows a relatively large influence on the pile response across various response levels and applied ground displacements (as seen in Figures 7a to 7j).

The crust layer shape factor α_C is a key uncertainty because it affects the pile response over a wide range of ground displacements when varied between its lower and upper bounds of α_C = 3 and 5, as shown in Figure 7. Note that in many guidelines and

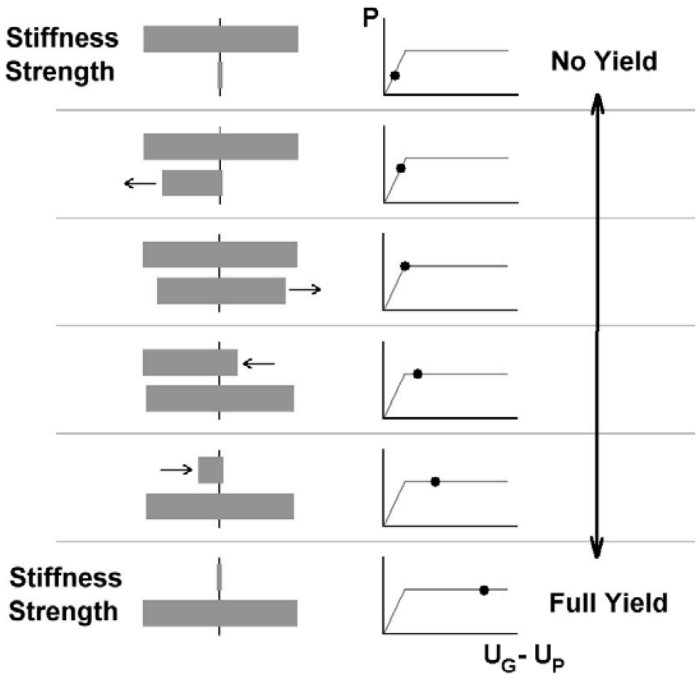


Figure 8. ***Conceptual illustration of transition from stiffness to strength controlled pile response and its relation to the mobilized load (deformation) of the soil spring relative to the yield level***

pseudo-static analysis procedures a value of α_C = 3 has been adopted, which is unconservative for piles in liquefying or laterally spreading soils. The value of α_C = 3 has been adopted based on the study of Broms (1964), in which active-pile-loading was considered. As illustrated in Figure 9a, under active-pile-loading the crust layer resists the pile deformation, and hence use of a lower-bound value for α_C ($\alpha_C = \alpha_{C-LB}$ = 3) would be conservative for this loading condition. However, piles in liquefying and laterally spreading soils predominantly undergo passive-pile-loading in which the crust provides a driving force for the pile deformation (Figure 9b). An upper-bound value for α_C ($\alpha_C = \alpha_{C-UB}$) would thus be the conservative choice for this loading mechanism. Recent experimental evidence from full-size tests on piles suggests that a value of α_C = 4.5 might be more appropriate for use in pseudo-static analysis of piles in liquefying or spreading soils (Cubrinovski et al., 2006).

There are inherent and very significant uncertainties associated with soil liquefaction during earthquakes and soil-pile interaction in liquefying soils. The extent of these uncertainties is reflected in the very wide ranges of values for the parameters of the liquefied soil in the pseudo-static model (β_L, α_L, S_r), including the effects of significant scatter in the empirical relationships used for their evaluation. It was noted in the analysis of the results presented in Figure 7 and schematic illustration in Figure 8 that there is a clear link between the sensitivity in the pile response (or influence level of a given parameter) and the mechanism of soil-pile deformation. Thus, model parameters affecting the soil stiffness more strongly influence the pile response when the relative displacement between the soil and the pile $|U_G - U_P|$ is smaller than the yield displacement of the soil, Δy. Conversely, model parameters affecting soil strength (ultimate pressure from the soil on the pile) more strongly influence the pile response after soil yielding has been initiated or when $|U_G - U_P| > \Delta y$. In order to examine the relative importance of the parameters of the liquefied layer, results from the pseudo-static analyses were plotted in $\Delta\phi/\phi_{ref}$ against $(U_G - U_P)/\Delta y$ diagrams in Figures 10a and 10b for β_L and, α_L and S_r respectively. Note that α_L and S_r affect the strength of the soil (ultimate pressure on the pile) and therefore the results for both parameters were plotted together in Figure 10b. Here, $\Delta\phi = |\phi_{UB} - \phi_{LB}|$ represents the difference between the peak curvatures computed in analyses using the upper and

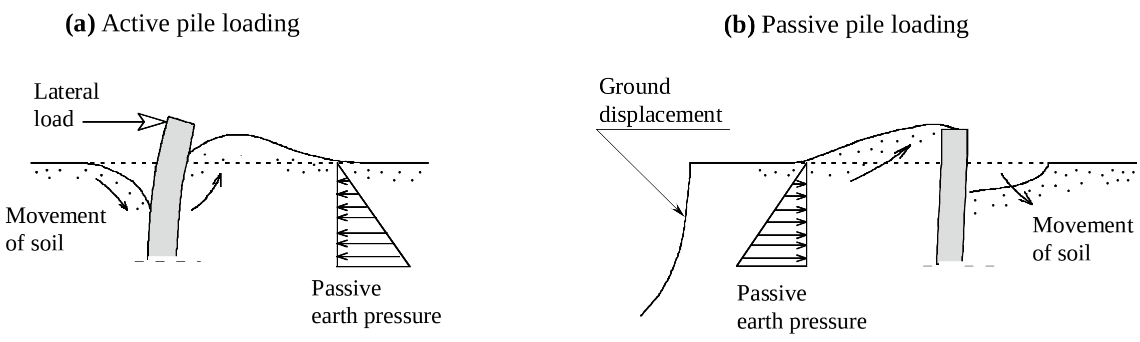
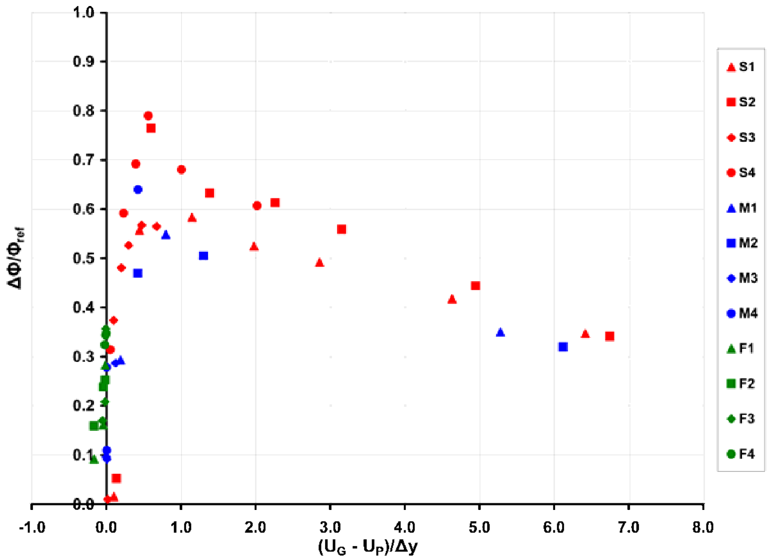
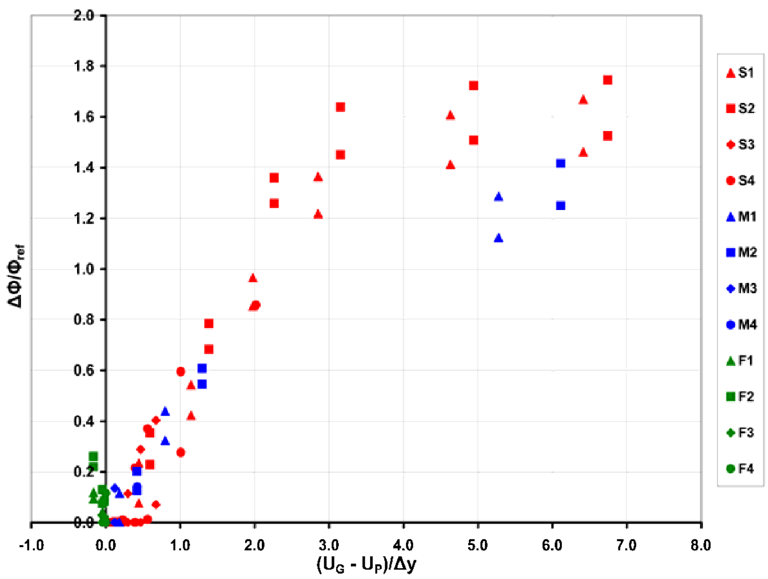


Figure 9: ***Schematic illustration of lateral loading of piles: (a) Active pile loading; (b) Passive pile loading***

lower bound values for a given parameter (essentially the size of the bar in the tornado charts), while ϕ_{ref} is the curvature computed in the analysis using reference model parameters (RM). Hence, the ratio $\Delta\phi/\phi_{ref}$ is a measure for the sensitivity in the pile response (curvature), and the plots indicate how the sensitivity changes with the yield ratio $(U_G - U_P)/\Delta y$. Note that $(U_G - U_P)/\Delta y$ = 1.0 indicates onset of yielding in the liquefied soil, at the pile head.



(a) Stiffness degradation factor for liquefied soil β_L



(b) Residual strength S_r and shape factor α_L for liquefied soil

Figure 10. ***Sensitivity of pile response (peak pile curvature) to parameters of liquefied soil for soil profiles with a crust layer of H_C = 1.5m obtained $_{in}$ lateral spreading PSA***

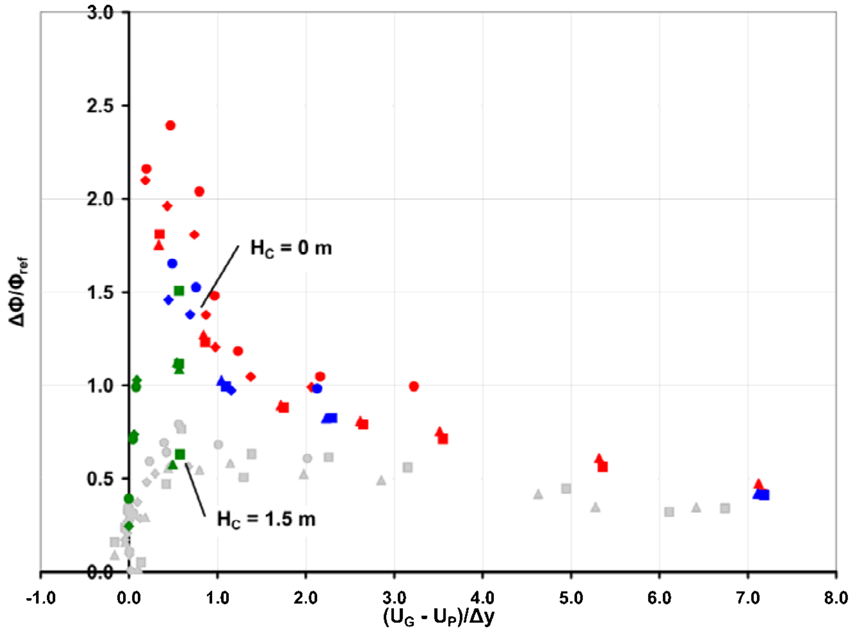
Figure 10a shows the sensitivity of the pile response to variations in the stiffness degradation factor β_L for soil profiles with a 1.5m thick crust, for all soil-pile systems considered (piles *S*, *M* or *F* embedded in profiles *P1*, *P2*, *P3* or *P4*). Here, different piles are represented by symbols of different colour, and the symbols shape indicates the soil profile. A very well defined relationship is seen irrespective of the wide range of soil and pile properties considered, thus clearly demonstrating the link between the influence level of β_L on the pile response (sensitivity) and loading (deformation) mechanism. Noting that a yield ratio of 1.0 corresponds to yielding of the soil (at the pile head at least), the plot shows that β_L is most important (most strongly affects the pile response) prior to soil yielding, when soil stiffness 'controls' the pile response. The sensitivity of the response to variations in β_L gradually decreases beyond yield ratios of 1.0, as more of the liquefied soil yields.

In reference to Figure 10b, the results for α_L and S_r also define a good relationship showing almost a linear increase in the influence of strength parameters (α_L and S_r) on the pile response with the yield ratio until a nearly stable maximum level has been reached at a yield ratio of about 3.0. Comparing Figures 10a and 10b, it is apparent that the sensitivity of the pile response is greater for β_L than for α_L or S_r when $|U_G - U_P|/\Delta y \leq 1.0$, whereas the reverse is true for $|U_G - U_P|/\Delta y > 2.0$. For $|U_G - U_P|/\Delta y > 3.0$, the sensitivity of the pile response to α_L or S_r is 3-4 times that of β_L.

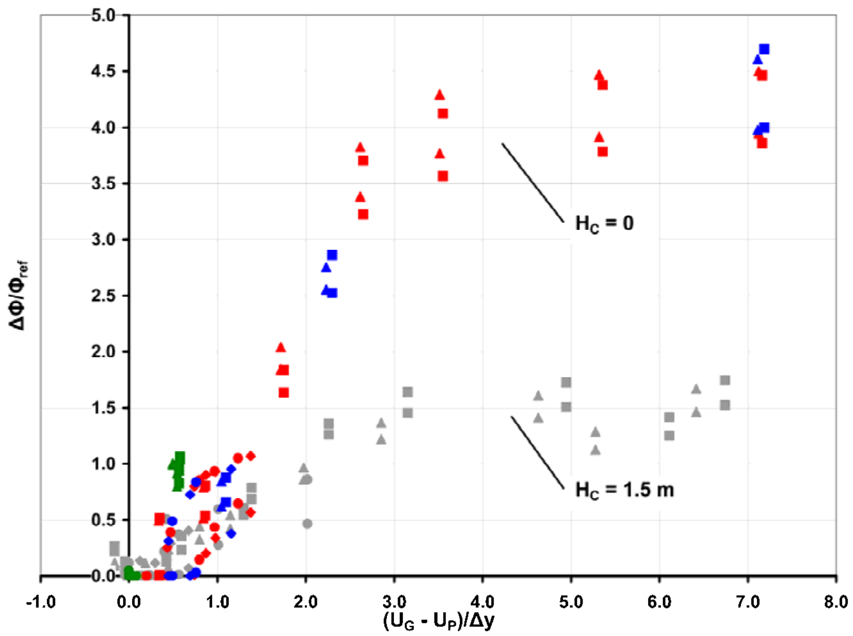
The same results are re-plotted in Figures 11a and 11b but now together with another set of results from analyses for profiles having no crust of non-liquefied soil at the ground surface, H_C = 0m. These plots thus depict how the sensitivity of the pile response to β_L (Figure 11a), and α_L and S_r (Figure 11b) is affected by the presence and thickness of the crust. Clearly, the sensitivity of the pile response to the parameters of the liquefied layer (β_L, α_L and S_r) decreases with an increasing thickness of the crust. However, the trends previously established in relation to the stiffness and strength 'controlled' (dominated) response and mechanism of soil-pile deformation remain unchanged.

3.7 HYPOTHETICAL CYCLIC SCENARIO

Using the same soil-pile systems (piles *S*, *M* or *F* embedded in soil profiles *P1*, *P2*, *P3* or *P4*), another series of PSA was carried out simulating a hypothetical cyclic scenario. In these analyses, model parameters and loading conditions considered were representative of the cyclic phase of the pile response during development and onset of liquefaction, except that inertial loads on the pile from a superstructure were ignored. In essence, these analyses were used to verify whether or not the findings obtained from the lateral spreading analyses are applicable to PSA evaluating the cyclic phase of the response. In the hypothetical cyclic scenario analyses, lower and upper bounds of β_L = 0.02 and β_L = 0.10 were adopted for the stiffness degradation factor for the liquefied soil, along with a reference value of $\beta_{L\text{-}ref}$ = 0.05. Note that this degradation in stiffness is much smaller than that adopted for lateral spreading reflecting the differences in excess pore pressures, and extent and severity of liquefaction between the cyclic phase and lateral spreading phase of the response. In the cyclic scenario PSA, ground displacements of 0.1, 0.2, 0.3 and 0.4m were applied to the pile implying average shear stresses in the liquefied soil in the range between 1% and 4%.



(a) Stiffness degradation factor of liquefied soil β_L



(b) Residual strength S_r and shape factor α_L for liquefied soil

Figure 11. ***Effect of crust thickness on sensitivity of pile response (peak pile curvature) to parameters of liquefied soil, observed in lateral spreading PSA***

Results from the hypothetical cyclic scenario PSA are presented in Figure 12 for β_L, α_L and S_r, and two crust thicknesses, H_C = 0m and 1.5m respectively. By and large, the trends observed for all parameters with regard to the sensitivity, deformation mechanism and crust thickness effects were the same as those reported for lateral spreading. The relationship for α_L and S_r shown in Figure 12 (for H_C = 1.5m) is very similar to the corresponding relationship for lateral spreading shown in Figure 10b. For the stiffness parameter β_L, on the other hand, the sensitivity of the response in the PSA simulating the cyclic phase was less than that observed in the lateral spreading PSA, and was particularly small for the case with H_C = 1.5m across the whole range of pile response.

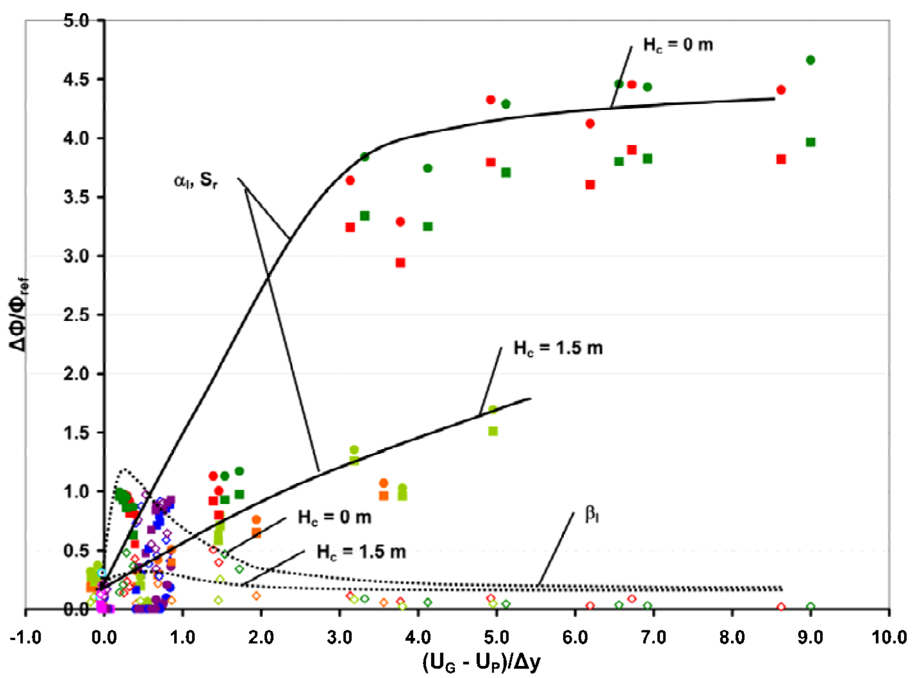


Figure 12. ***Sensitivity of pile response (peak pile curvature) to parameters of liquefied soil, for hypothetical cyclic scenario (ignored inertial loads) including effects of crust thickness (H_C = 0m and H_C = 1.5m); solid lines indicate sensitivity to α_L and S_r, while dashed lines are for β_L***

3.8 CYCLIC PHASE PSA: COMBINED KINEMATIC AND INERTIAL FORCE DEMANDS

One of the key issues in the PSA when used for evaluating the peak response of the pile during the cyclic phase is how to combine the kinematic loads due to lateral ground displacements and the inertial loads due to vibration of the superstructure. The peak cyclic ground displacement and superstructure inertial force are transient conditions occurring momentarily during the course of strong shaking. They may or may not occur at the same instant, hence there is no clear and simple strategy how to combine these loads in the PSA. It has been suggested that the phasing of the kinematic and inertial demands varies, and depends primarily on the natural frequency of the superstructure and soil deposit (Tamura and Tokimatsu, 2005). Recently, Boulanger et al. (2007) suggested a simplified expression allowing for different combinations of kinematic and inertial loads on the pile while accounting for the period of the ground motion. As commonly acknowledged for pseudo-static approaches for analysis of seismic problems, the load combination producing the critical (peak) pile response in liquefying soils cannot be predicted with any high degree of certainty. The aim of the presented analyses herein was not to determine how best to approximate the critical pseudo-static demand on the pile, but rather to investigate the influence of this modelling decision (kinematic-inertial load combination) on the predicted pile response.

In this series of analyses, a horizontal force acting at the pile head representing the load on the pile from the superstructure was applied in addition to the lateral ground displacement. In total, 20 load combinations were considered for each soil-pile system: five inertial loads (lateral force at pile head) corresponding to horizontal accelerations of 0.1, 0.2, 0.3, 0.4 and 0.5g, and four lateral ground displacements of 0.1, 0.2, 0.3 and 0.4m at the ground surface. Depending on the base layer (soft clay or dense sand), different axial capacities were adopted for the piles resulting in substantially different inertial loads. For example, the inertial loads for case *S-P1* were in the range between 75 and 375 kN (for 0.1g and 0.5g respectively) while the respective inertial loads for Case *S-P2* were 300 and 1500 Kn.

Parametric sensitivities

The introduction of an inertial force at the pile head, in addition to the kinematic soil demands adds another dimension to the already complex problem. For a given scenario, this force may change the fundamental mechanism of soil-pile interaction, increase the severity of the damage suffered by the pile, and alter the influence other parameters have on the predicted pile response. Detailed discussion on the combined kinematic and inertial effects on the pile response may be found in Haskell (2009). Herein the influence of the inertial force on the sensitivity of the pile response to parameters of the liquefied soil is examined in a fashion similar to that presented in the preceding sections. To simplify the problem, all cases that resulted in an unrealistic response (e.g. pile displacements significantly greater than the cyclic ground displacement) or unacceptable level of pile damage (well in excess of the ultimate level, ϕ_u) were not considered. With reference to the induced level of pile displacement and consequent soil-pile interaction mechanism, only the mechanism of 'stiff-pile-behaviour' illustrated in Figure 13 where $U_G > U_P$ and $(U_G - U_P) > \Delta y$ is associated with sensitivity of the pile response to variation in the parameters of the liquefied soil. Note that, in the cyclic PSA with combined kinematic and inertial loads, the 'stiff-pile-behaviour' mechanism and hence parametric sensitivity was observed for only few of the examined soil-pile systems and loading conditions, as summarized in Table 3.

Table 3. Acceleration levels associated with 'stiff-pile-behaviour' in cyclic PSA with combined kinematic and inertial force demands

Crust Thickness, H_C (m)	*S-Pile*				*M-Pile*				*F-Pile*			
	S-P1	*S-P2*	*S-P3*	*S-P4*	*M-P1*	*M-P2*	*M-P3*	*M-P4*	*F-P1*	*F-P2*	*F-P3*	*F-P4*
0.0	≤0.5g	≤0.2g*)	≤0.3g	-	≤0.5g	≤0.2g	≤0.3g	-	-	-	-	-
1.5	-	≤0.2g	-	-	-	≤0.2g	-	-	-	-	-	-

*) For example, for *S-P2*, "stiff-pile-behaviour" was obtained for inertial loads corresponding to accelerations of less then 0.2g.

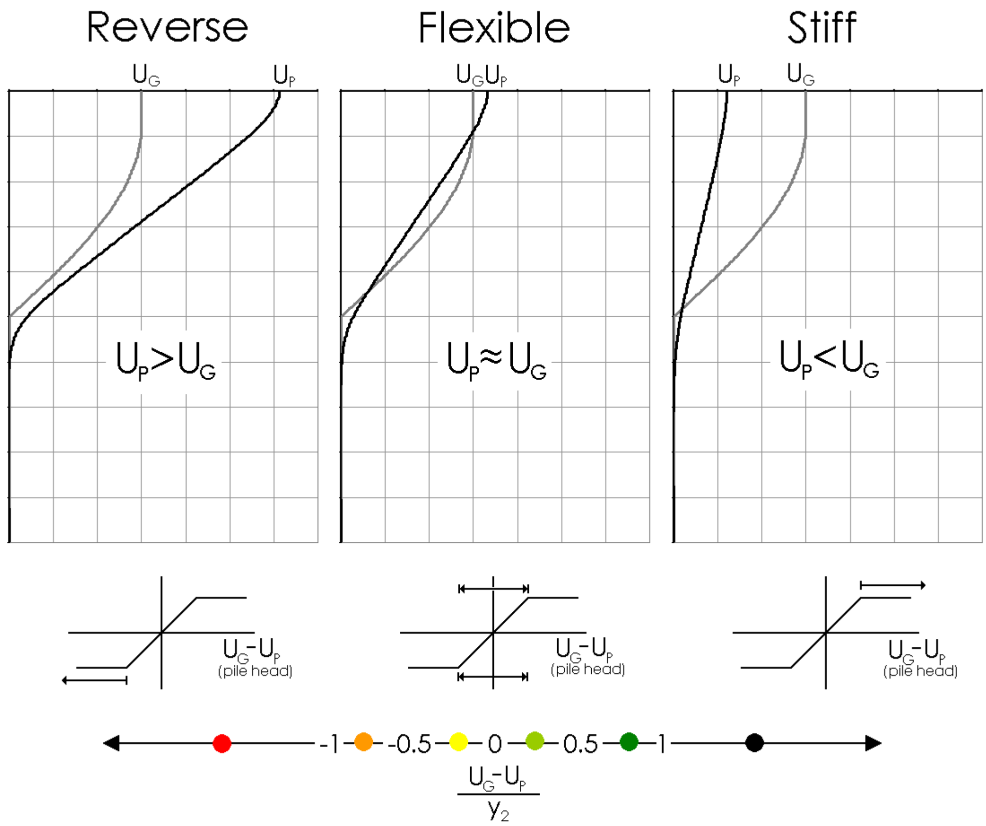
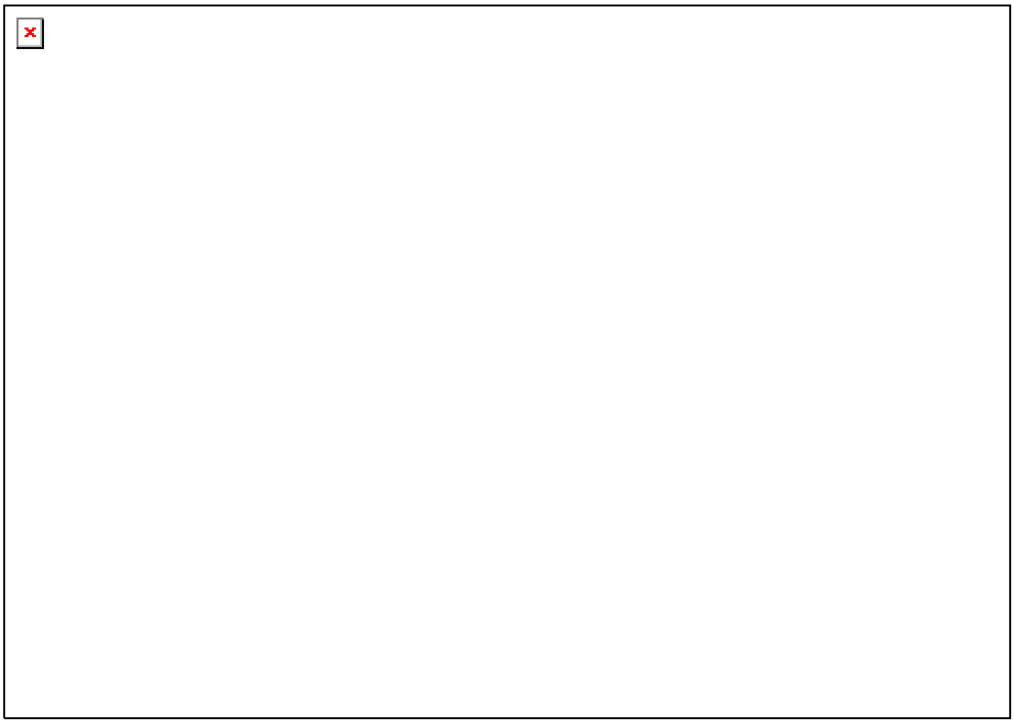
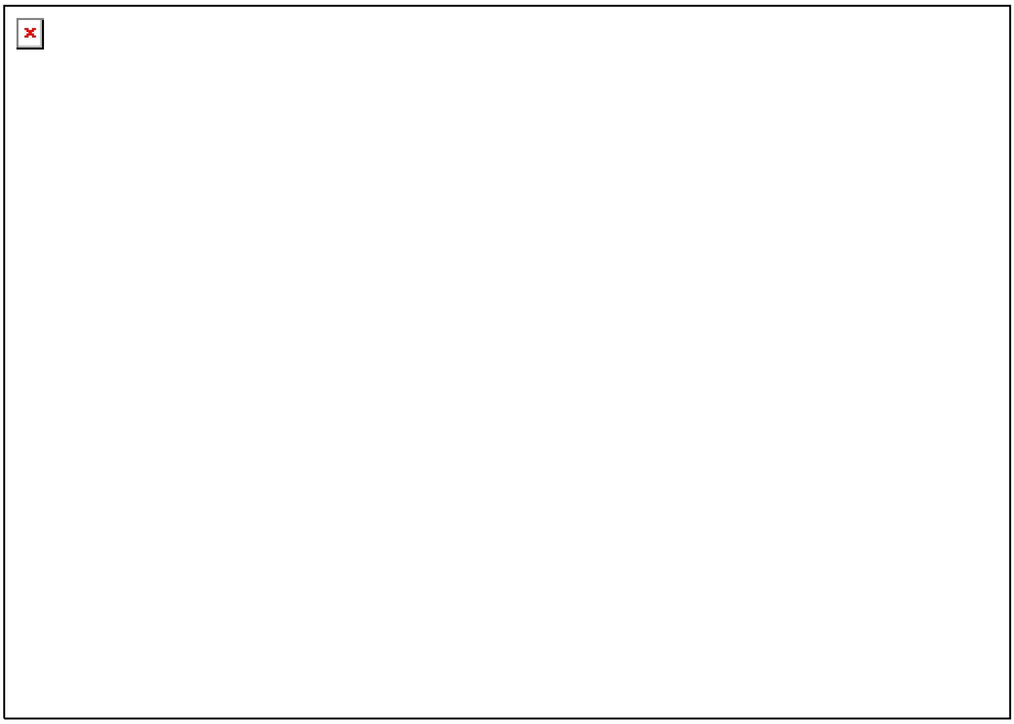


Figure 13. Schematic illustration of 'reverse', 'flexible-pile-behaviour' and 'stiff-pile-behaviour' based on relative displacements between the soil and the pile

Results from analyses of the soil-pile system *S-P1* are shown in Figures 14a and 14b for the parameters β_L and α_L respectively. The application of and increase in the inertial force reduces the relative soil-pile displacement making the response more flexible and hence decreasing the yield ratio. It is also apparent from the plots that an increase in the inertial force reduces the sensitivity of the pile response to the parameters of the liquefied soil. This effect on the sensitivity of the pile response to the liquefied soil parameters is analogous to that of the non-liquefied crust. Indeed, where the yield ratio is greater than one, equivalent crust and inertial forces have an identical effect on the sensitivity of the response to parameters of the liquefied soil.



(a) Stiffness degradation factor of liquefied soil β_L



(b) Residual strength S_r and shape factor α_L

Figure 14. Sensitivity of pile response (peak pile curvature) to parameters of liquefied soil, for cyclic scenario (combined kinematic and inertial force demands)

This outcome is intuitively expected, because the inertial force and the resultant lateral load from the crust act at nearly identical locations (at or near the pile head), and hence when the magnitudes of these two loads are similar, their effects on the pile response should also be similar. There is one important difference between these two loads however. Whereas the size of the inertial force is predetermined as an input in the PSA, the magnitude of the lateral force from the crust on the pile depends on the pile response or computed relative displacement ($U_G – U_P$). The general tendency in the effects of stiffness and strength parameters on the pile response and their relation to pre-yield or post-yield deformational behaviour was also evident for these (stiff-pile-behaviour) cases with combined inertial and kinematic loads. The sensitivity level of the response was also similar to that observed in the lateral spreading PSA.

3.9 SUMMARY AND CONCLUSIONS

Results from comprehensive series of parametric analyses have been used to examine and quantify the sensitivity of the pile response to various model parameters, and hence, to identify critical uncertainties in the pseudo-static analysis. Key findings from the study can be summarized as follows:

- Sensitivity of the pile response to parameters of the liquefied soil is clearly related the soil-pile interaction mechanism or load (deformation) of the soil spring relative to the yield level. For small relative displacements $|U_G – U_P|$, the pile response is most sensitive to the stiffness degradation factor (β_L). As the relative displacement increases and yielding in the soil occurs, the response becomes more sensitive to the strength of the soil (parameters α_L and S_r).
- The stiffness degradation factor (β_L) is most important and shows greater influence on the pile response than the strength parameters for yield ratios of up to 0.5-1.0. Its influence on the pile response gradually decreases as more soil springs in the liquefied soil yield.
- The influence of strength parameters on the pile response gradually increases with the relative displacement between the soil and the pile, and reaches the maximum level at yield ratios of about 3 or higher. For yield ratio of 1.0 (at the initiation of soil yielding) the effects of strength parameters (α_L and S_r) on the pile response are already at the same level or greater than those of β_L. At high yield ratios (≥ 3), the influence of the strength parameters on the pile response is substantially higher than that of β_L (3 to 10 times greater sensitivity).
- Sensitivity of the pile response to the parameters of the liquefied layer (β_L, α_L and S_r) decreases with the thickness (or load contribution) of the crust.
- The above conclusions are applicable to pseudo-static analyses of piles for both cyclic ground displacements and lateral spreading displacements.
- For cyclic liquefaction with combined kinematic and inertial force demands, it was found that parametric sensitivity is an issue only for stiff-pile-behaviour where the pile resists the combined lateral loads and exhibits considerably smaller displacement than the applied ground displacement. For these cases, the effect of inertial load on the pile response is analogous to that of a non-liquefied crust; an increase in the inertial load decreases the sensitivity of the pile response to parameters of the liquefied soil.
- The shape factor α_C is a key uncertainty associated with the ultimate load on the

pile from a crust of non-liquefied soil at the ground surface. The value of this factor needs to be selected in conjunction with the anticipated role of the crust layer in the loading mechanism (active-pile-loading vs. passive-pile-loading).

- The sensitivity of the pile response to parameters of the base layer and initial soil stiffness is negligibly small.
- The representative SPT blow count (soil characterization parameter) is a significant uncertainty in the analysis since it affects multiple model parameters (stiffness and strength) and shows relatively large influence on the pile response across a wide range of load (response) levels.

The above conclusions are generally applicable to pseudo-static methods for analysis of piles in liquefying soils even though specific parameters and details of the model may differ from those adopted in this study.

Acknowledgements

The authors would like to acknowledge the financial support provided by the Earthquake Commission (EQC), New Zealand.

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CHAPTER 4: THE EFFECT OF SHEAR STRENGTH NORMALISATION ON THE RESPONSE OF PILES IN LATERALLY SPREADING SOILS

Cubrinovski, M., Haskell, J. and Bradley B. (2009). Earthquake Geotechnical Engineering Satellite Conference. XVIIth International Conference on Soil Mechanics & Geotechnical Engineering 2-3. 10. 2009, Alexandria, Egypt

ABSTRACT

In the simplified pseudo-static analysis of piles, the ultimate lateral pressure from the liquefied soil is commonly approximated based on the residual strength of liquefied soils. This strength does not have sound theoretical basis, but rather is estimated from one of several empirical relationships between the residual strength and penetration resistance. The two empirical relationships adopted in this study, even though originating from the same database, result in substantially different strength profiles (ultimate lateral pressures on the pile) throughout the depth of the liquefied layer. Series of analyses were conducted to investigate the effects of strength normalisation on the pile response predicted by the pseudo-static analysis. It was found that effects of strength normalisation can be quite significant and that they depend on the relative stiffness of the pile and thickness of a non-liquefiable crust at the ground surface.

Keywords: Liquefaction, lateral spreading, pile, pseudo-static analysis

4.1 INTRODUCTION

The response of piles in liquefying deposits during earthquakes is very complex involving rapidly varying dynamic loads and significant reduction in soil stiffness and strength caused by liquefaction. During lateral spreading of liquefied soils, the piles are subjected to large kinematic loads due to lateral movement of the spreading soil and comparatively smaller inertial loads from the diminishing vibration of the superstructure. As illustrated schematically in Figure 1, the liquefied soil and an overlying crust at the ground surface provide driving forces for the pile displacement in the direction of spreading, while the base soil resists the pile movement. In the pseudo-static method of analysis, a relatively simple soil-pile model based on this mechanism is used to estimate the maximum deformation of the pile and its consequent damage due to spreading.

The simplified pseudo-static analysis is burdened by significant uncertainties regarding the characterization of lateral loads on the pile and properties of the adopted soil-pile model. The uncertainties and difficulties in the modelling are particularly pronounced for the parameters of the liquefied soil, such as stiffness, strength and displacement of the liquefied soil. This paper focuses on one particular aspect in the modelling of the lateral pressure from the liquefied soil and its effects on the pile response. Namely, the approximation of the ultimate pressure from the liquefied layer on the pile based on the residual strength of liquefied soils. A couple of well-known empirical relationships are available for estimating the residual strength of liquefied soils, one using non-normalised residual strength (S_r) and the other using normalised residual strength (S_r/σ'_{vo}). The key difference in the context of pile analysis is that these two relationships suggest very different distributions of strength (ultimate lateral pressure from the soil on the pile) throughout the depth of the liquefied layer. In this study, series of pseudo-static analyses were conducted to examine and quantify the effects of stress normalisation on the pile response predicted by the pseudo-static analysis. Models with different crust thickness and piles of different stiffness were considered in the analyses.

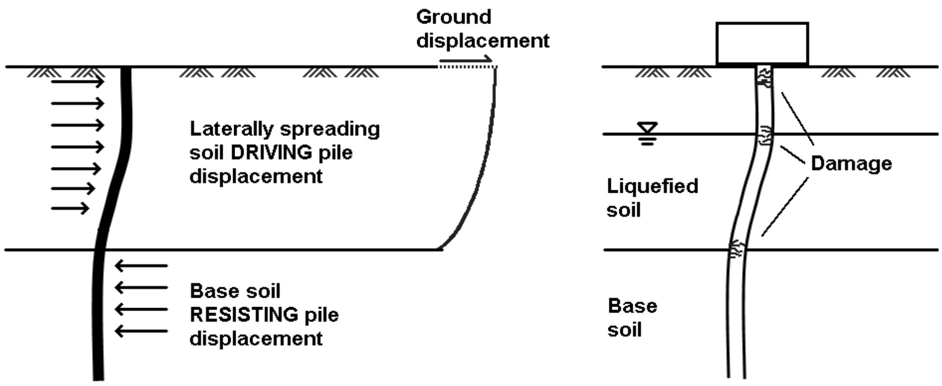

Figure 1. Schematic illustration of lateral loads on piles due to spreading of liquefied soils and consequent damage

4.2 PSEUDO-STATIC ANALYSIS OF PILES

The pseudo-static method of analysis provides a practical engineering approach for seismic assessment of piles based on routine computations and use of relatively simple models. It aims at estimating the peak response of the pile (maximum strains or curvature of the pile) due to earthquake shaking or lateral spreading under the assumption that complex dynamic loads can be idealized as static actions. Generally, two approaches are used for pseudo-static analysis of piles subjected to lateral spreading: force-based methods and displacement-based methods. These two approaches differ in the way in which the lateral load on pile due to ground movement (kinematic load) is considered.

In force-based methods, an equivalent static load representing the pressure from the laterally spreading soil is applied to the pile. For a typical three-layer configuration with a liquefied layer sandwiched between a non-liquefied surface layer (crust) and a non-liquefied base layer, the lateral earth pressures from the crust and liquefied layer are estimated and applied as driving loads (pushing the pile in the direction of spreading), as shown in Figure 2a. One serious deficiency of this approach is that it ignores the dependence of the magnitude of the mobilized lateral soil pressure on the pile response (or relative displacement between the soil and the pile). In this context, the displacement-based methods offer more rigorous modelling that is compatible with the mechanism of soil-pile interaction. In this approach lateral ground displacements (representing free field ground movement) are applied at the free end of soil springs attached to the pile, as illustrated in Figure 2b. In this case, the forces that develop in the soil springs are compatible with the relative displacement between the soil and the pile, and hence, the mobilized lateral soil pressure is compatible with the induced pile response. The displacement-based approach allows scrutiny of the behaviour of piles over the entire range of deformation, from elastic (small lateral loads) to failure (large lateral loads), and therefore was adopted in this study. Note however that, in principle, the conclusions with regard to the effects of shear strength normalisation on the pile response (the subject of this study) are applicable to both displacement-based and force-based methods.

A typical beam-spring model representing the soil-pile system in the simplified pseudo-static analysis is shown in Figure 3. Since a key requirement of the analysis is to estimate the inelastic deformation and damage to the pile, simple but non-linear load-deformation relationships are used for the soil-pile model. The pile is modelled using a series of beam elements with a trilinear moment-curvature relationship, while the soil is represented by bilinear springs in which degraded stiffness and strength of the soil are employed to account for effects of nonlinear behaviour and liquefaction. Since the behaviour of piles in liquefying soils is extremely complex, involving very large and rapid changes in soil stiffness, strength and lateral loads on the pile, one of the key questions in the implementation of the pseudo-static analysis is how to select appropriate values for these parameters in the equivalent static analysis. In other words, what are the appropriate values for β, $p_{L\text{-}max}$, U_G and F in the model shown in Figure 3? While discussion on their determination, uncertainties in these parameters and the sensitivity of the pile response to their variation can be found in Cubrinovski et al. (2009), Cubrinovski and Bradley (2008) and Haskell et al. (2009) respectively, the attention focus here is one particular aspect in the modelling of the lateral pressure from the liquefied soil on the pile.

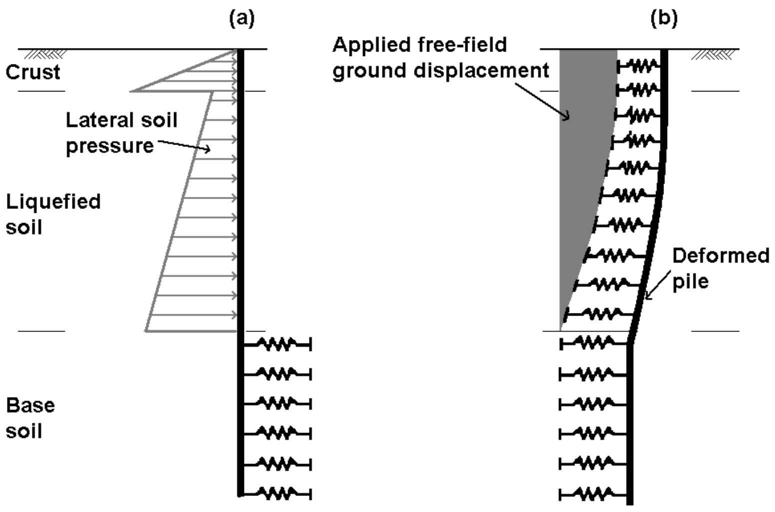

Figure 2. Pseudo-static methods for analysis of piles: (a) Force-based approach; (b) Displacement-based approach

4.3 LATERAL PRESSURE FROM LIQUEFIED SOILS

The significant uncertainties associated with the stiffness and strength of liquefied soils result in a large anticipated variation of the bilinear *p*-δ relationship for the liquefied soil, as illustrated in Figure 4. The ultimate lateral pressure from the liquefied soil ($p_{L\text{-}max}$) is often approximated based on the residual (shear) strength of liquefied soils (S_r) as

$$p_{L-\max} = \alpha_L S_r \tag{1}$$

where α_L is a factor that accounts for the volume of soil contributing to the generation of soil pressure on the pile (equivalent to the wedge-mechanism concept).

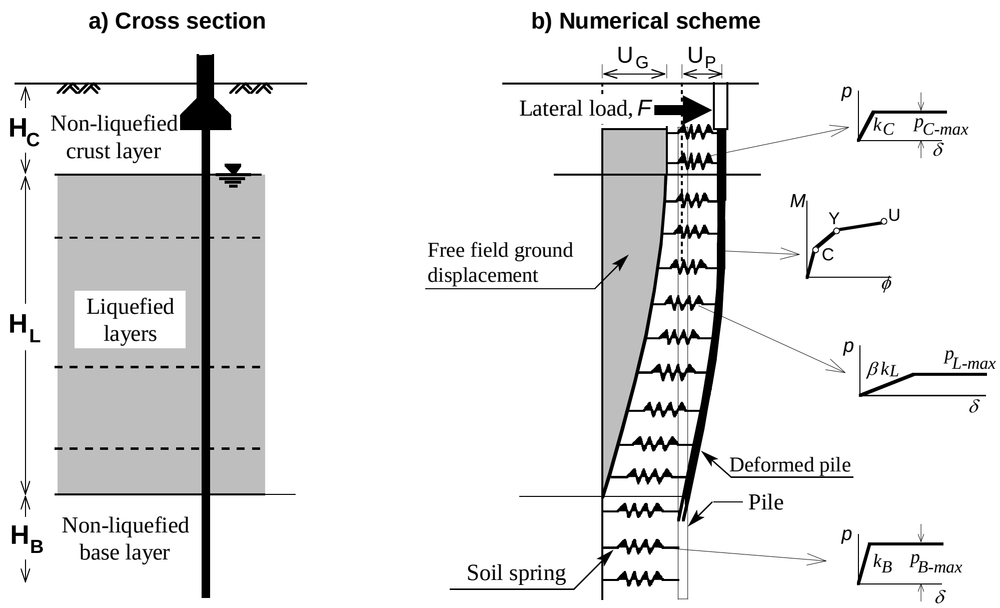

Figure 3. Typical beam-spring model for simplified pseudo-static analysis of piles

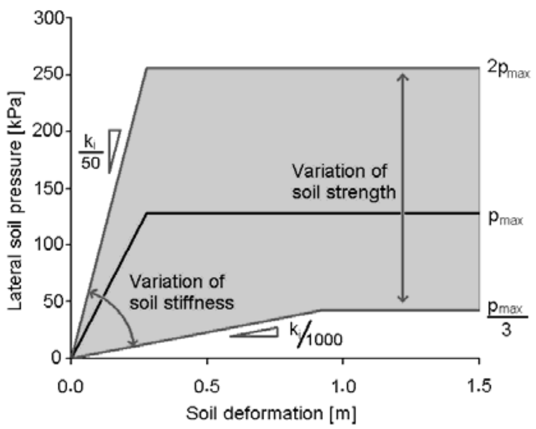

Figure 4. Schematic illustration of variation in the p-δ relationship for the liquefied soil

There are several empirical relationships between the residual strength of liquefied soils and penetration resistance established using back-calculations from liquefaction case histories. Based on an earlier work by Seed (1987), Seed and Harder (1990) proposed an empirical relationship between S_r and SPT blow count $(N_1)_{60cs}$, shown in Figure 5a. The relationship encompasses data of roughly 20 case histories and is characterized by considerable scatter. For example, for a normalised equivalent-sand blow count of $(N_1)_{60cs}$ = 10, the residual strength takes values between 5 kPa and 25 kPa. Using the same case history data, Olson and Stark (2002) proposed an alternative relationship between the residual strength and SPT resistance in which a normalised residual strength (S_r/σ'_{vo}) is correlated with $(N_1)_{60}$, as shown in Figure 5b. Here, the shear strength of the liquefied soil at depth z is normalised by the respective effective overburden stress, $\sigma'_{vo}(z)$. Recently, Idriss and Boulanger (2008) reinterpreted the same data set and proposed a pair of empirical relationships, discriminating between cases in which voids ratio redistribution (loosening of the soil during the liquefaction process) occurs or not. Again, they presented their relationships in two forms, non-normalised S_r-$(N_1)_{60}$, and normalised (S_r/σ'_{vo})-$(N_1)_{60}$.

The normalisation (or not) of shear strength of liquefied soils is an unresolved issue and recommendation of one method in preference to the other is beyond the scope of this paper. Rather, this study investigates the effect of this normalisation on the response of piles predicted using the simplified pseudo-static analysis.

4.4 INVESTIGATED SOIL-PILE MODELS

Comprehensive series of parametric analyses were conducted for different soil profiles and piles encompassing a wide range of soil properties (very loose to medium dense liquefied soil; very soft to very dense base layer) and pile characteristics (flexible to stiff piles). Here, analyses and results for one of these soil profiles are presented in order to illustrate the effects of strength normalisation on the pile

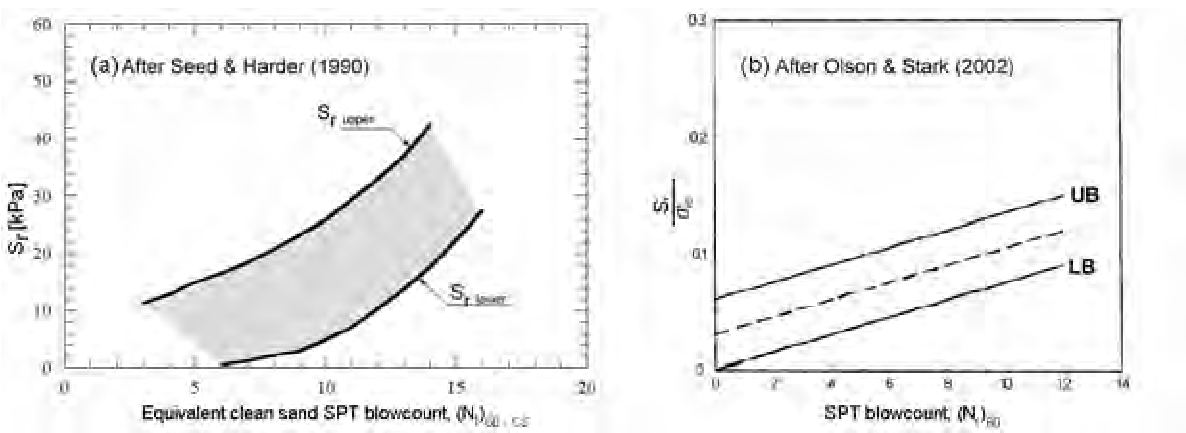

Figure 5. Empirical relationships between residual strength of liquefied soil and penetration resistance: (a) Non-normalised, Seed and Harder (1990); (b) Normalised, Olson and Stark (2002)

response. As shown in Figure 6, a 20 metre-long pile is embedded in a deposit consisting of two layers, a loose sand layer (N =5) overlying a non-liquefiable base layer of stiff gravel. Both layers have a thickness of 10 m. Assuming that the soil above the water table acts as a crust of non-liquefiable surface soil, five different scenarios were adopted for the location of the water table between z = 0 and 2 m depth, defining a crust of thickness of H_C = 0, 0.5, 1, 1.5 and 2 m respectively. The remaining part of the loose sand layer below the water table defined the thickness of the liquefiable layer, $H_L = 10 - H_C$ = 10, 9.5, 9, 8.5 and 8 m respectively. To account for the effects of relative pile stiffness on the response, three different piles with diameters of 400, 800 and 1200 mm were considered as representatives of a relatively flexible (*F-Pile*), intermediate *(M-Pile*) and relatively stiff pile *(S-Pile*) respectively. Trilinear moment-curvature relationships of actual reinforced concrete piles were adopted for the piles. In total 15 computational models were considered, with five different thicknesses of the crust and three different piles.

Details about determination of model parameters including range of realistic values, best-estimates or reference values, and effects of uncertainties on the pile response are given in Haskell et al. (2009). In the analyses of the strength normalisation effects presented herein, models with reference values of the parameters were used. For example, for the parameter α_L a reference value of α_L = 3 was adopted from a range of expected realistic values from 1 to 6. For each of the 15 computational models introduced above, lateral spreading displacements (U_G) ranging from 0.1 to 2.0 m were applied as input in the pseudo-static analysis. For each case considered, two analyses were performed, one using non-normalised strength for the liquefied soil (as proposed by Seed and Harder, 1990) and the other using normalised strength for the liquefied soil (as proposed by Olson and Stark, 2002). The resulting difference in the shear strength profiles (and respective distribution of ultimate pressure on the pile) for the two methods is schematically illustrated in Figure 7.

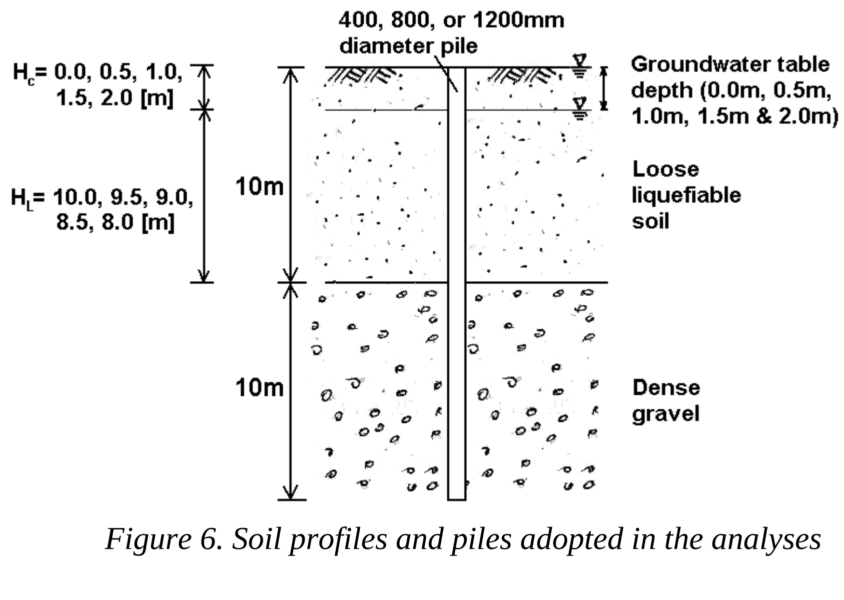

Figure 6. Soil profiles and piles adopted in the analyses

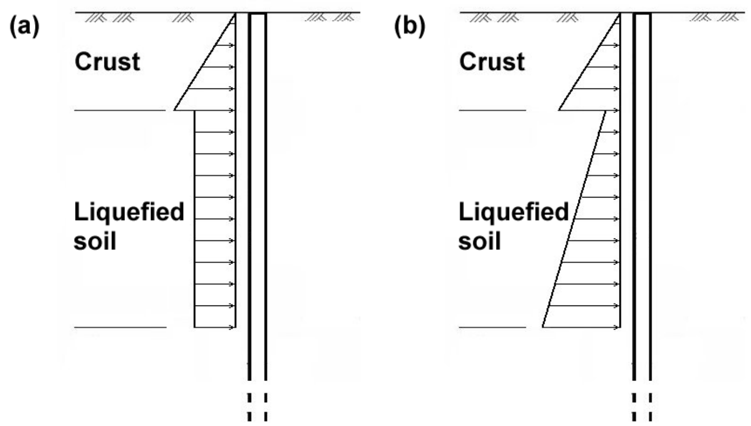

Figure 7. *Distribution of residual soil strength (ultimate lateral pressure from the liquefied soil) for the methods based on: (a) Non-normalised strength, S_r; (b) Normalised strength, (S_r/σ'_{vo})*

4.5 ANALYSIS RESULTS

Results of the analyses are presented in terms of the computed peak pile curvature along the length of the pile, because this curvature indicates both the size of the pile response and the level of damage to the pile. Figure 8 comparatively shows the peak pile curvatures computed in the analyses using normalised residual strength (S_r/σ'_{vo}) and non-normalised strength (S_r) for the liquefied soil. There are three phases in the relationship shown in this idealised plot that are directly related to the load-deformation mechanism of the pile. Before initiation of soil yielding (from the origin to point *A*), both analysis methods produce identical results, because in this phase the pile response is not affected by $p_{L\text{-}max}$ (strength S_r). However, once soil yielding is initiated (point *A* in Figure *8*) the response deviates from the 1:1 relationship. In the second phase (from point *A* to point *B*), the rate of increase of curvature with applied ground displacement reduces in the analysis using normalised residual strength as compared to the analysis using non-normalised residual strength. In other words, for a given ground displacement, a smaller curvature is computed in the analysis using normalised residual strength in the calculation of $p_{L\text{-}max}$. To clarify this response we need to compare the process of soil yielding for the two methods. In the analysis with normalised residual strength, soil yielding first occurs at the top of the liquefied layer as this is the location where the yield stress in the liquefied soil is the lowest, as apparent in Figure 7. The soil yielding effectively limits the lateral load from the soil on the pile, resulting in a smaller pile displacement (U_P) and consequently, larger relative displacement between the soil and the pile, $\delta = U_G - U_P$. This in turn causes propagation of the soil yielding front from the top of the liquefied layer towards the base of this layer. Eventually, the relationship levels off at point *B*, once soil yielding has been triggered throughout the entire depth of the liquefied layer and the maximum lateral load from the liquefied soil has been mobilized. The same process applies to the analysis with non-normalised strength, except that it starts and ends at larger ground displacements and pile curvatures.

Comparative plots of computed peak pile curvatures are shown in Figures 9a, 9b and 9c for piles with diameters of 400, 800 and 1200 mm respectively. For each case, results for five different thicknesses of the crust (H_C = 0, 0.5, 1, 1.5 and 2 m) are presented. For the flexible pile (*F-Pile*), very large effects of normalisation are seen for the case without crust. For this case, the ultimate pile curvature (ϕ_U) was exceeded in the analysis using non-normalised strength, whereas in the corresponding analysis using normalised strength, the computed curvature was below the cracking level. In other words, the normalisation changed the pile performance from ‘failure’ to ‘no damage’. Much smaller effects of normalisation are seen for a deposit with a 0.5 m thick crust, while no effects are seen for crusts with thicknesses of 1, 1.5 and 2 m. Effects of normalisation in reducing the peak pile curvature are also evident for the *M-Pile* and *S-Pile* for all cases with a crust thickness below 2 m.

Clearly, the effects of normalisation on the pile response predicted by the pseudo-static analysis could be significant, and they depend both on the properties of the pile and thickness of the crust layer. To summarize these effects, the ratio of curvatures ϕ_A/ϕ_U is plotted against the thickness of the crust (H_C) in Figure 10. Here, ϕ_A and ϕ_U denote the peak pile curvature at which effects of normalisation start to influence the pile response (corresponding to point *A* in Figure 8) and the ultimate curvature of the pile respectively. The plot basically indicates whether or not the normalisation will affect the pre-failure response of the pile, as a function of pile stiffness and crust thickness. For example, for relatively flexible piles (*F-Pile*), the strength normalisation would affect the response of the pile only if the crust thickness is less than 0.9 m. On the other hand, for relatively stiff piles (*S-Pile*), the normalisation will affect the pre-failure response of the pile when the thickness of the crust is less than 1.75 m. In other words, the load from the crust would practically govern the pile response and obscure the effects of strength normalisation for the liquefied soil when H_C > 1.75m. Hence for these cases, the normalisation of the strength of the liquefied soil is not an issue.

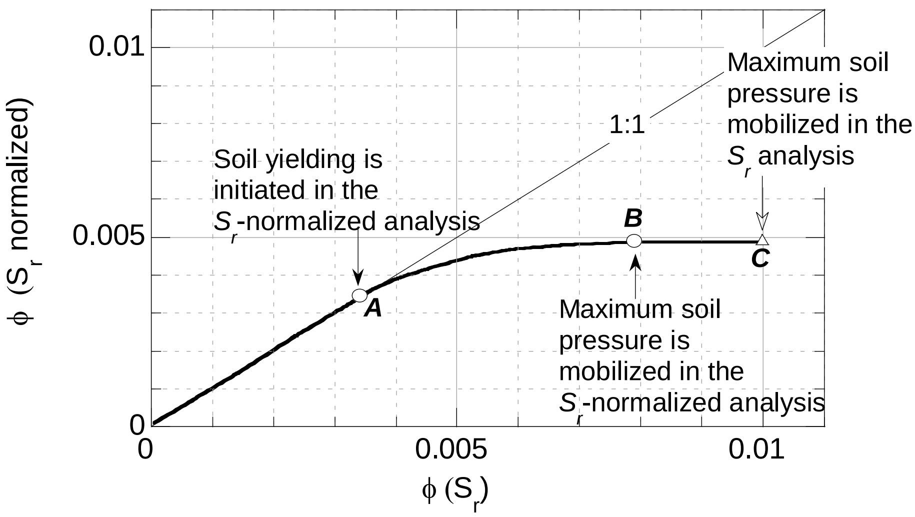

Figure 8. Typical relationship between peak pile curvatures computed in the analyses with non-normalised (S_r) and normalised strength (S_r/σ'_{vo})

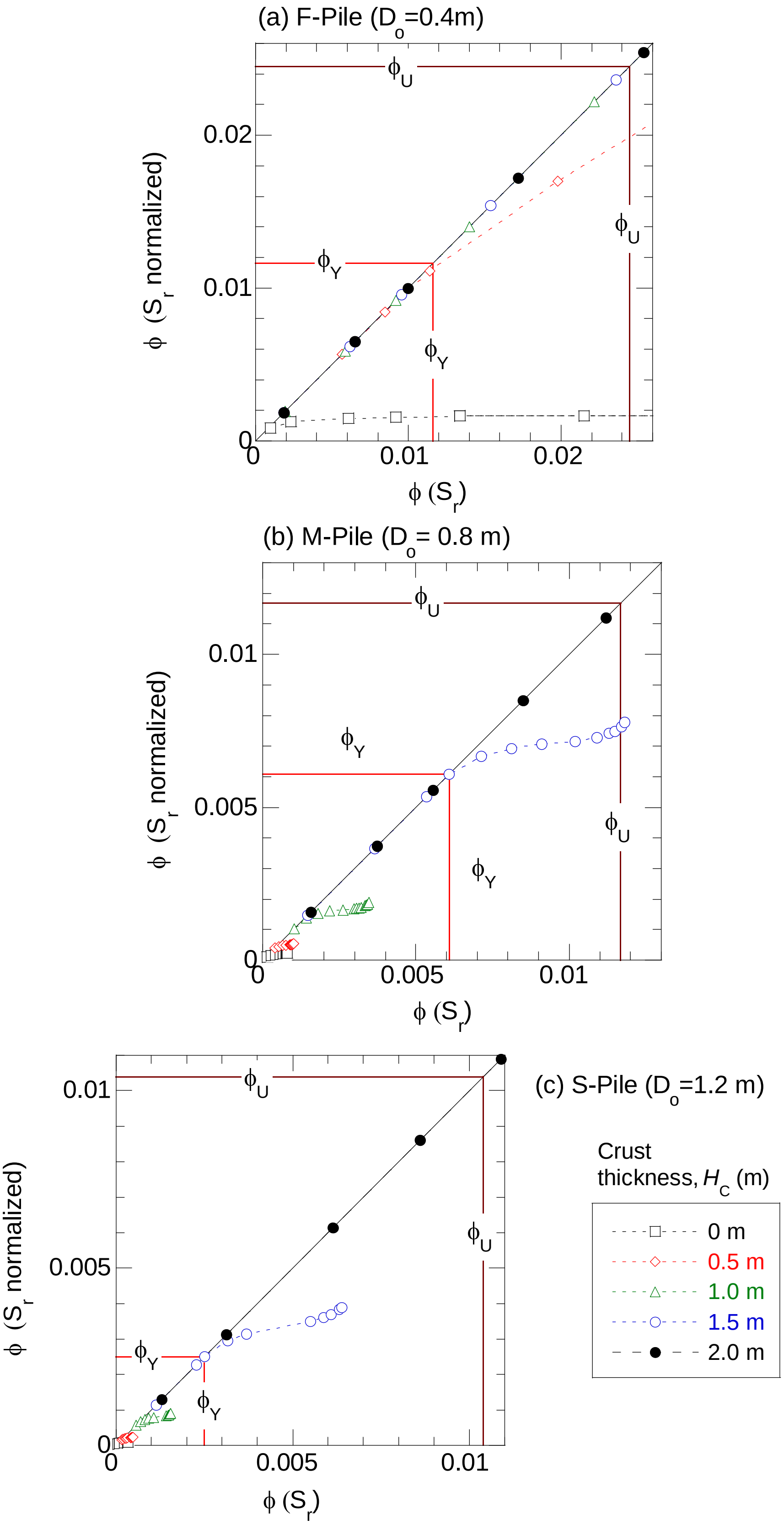

Figure 9. Comparison of peak pile curvatures computed in the analyses with non-normalised (S_r) and normalised strength (S_r/σ'_{vo}): (a) F-Pile; (b) M-Pile; (c) S-Pile

To illustrate the magnitude of the normalisation effects on the pile response, Figure 11 shows the ratio of the peak pile curvatures computed by the two analysis methods as a function of the applied ground displacement. The figure depicts the amount of reduction in the pile response due to strength normalisation for different pile stiffness and thicknesses of the crust. The reduction is very pronounced (70-90%) for deposits without crust, and is still significant (30-40%) for medium-stiff to stiff piles with crusts of up to 1.5 m thick.

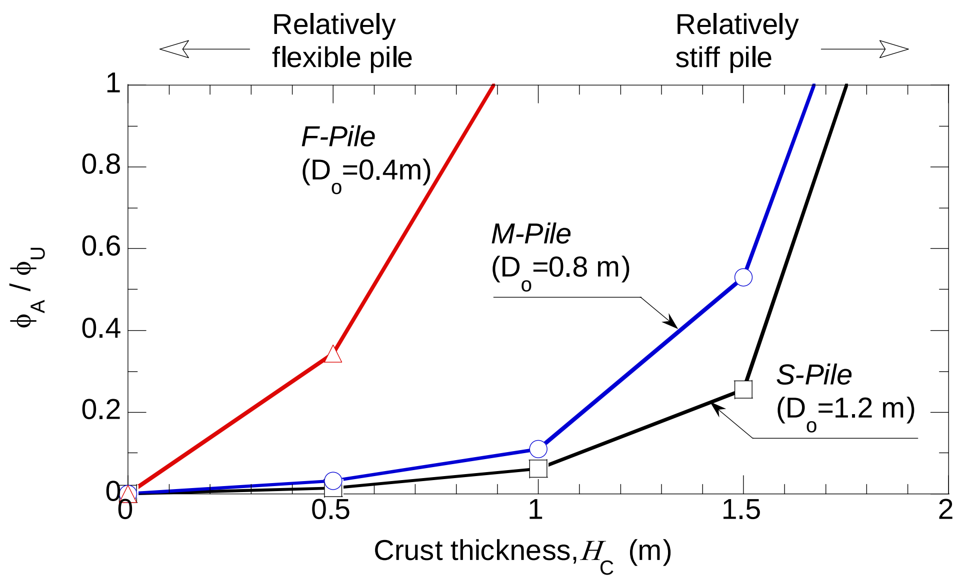

Figure 10. *Illustration of curvature levels (ϕ_A/ϕ_U) at which strength normalisation starts to influence the pile response, as a function of pile stiffness and crust thickness*

4.6 CONCLUSIONS

Effects of soil strength normalisation on the pile response predicted by a simple pseudo-static analysis have been investigated in this paper. Two well-known empirical relationships for residual strength of liquefied soils, one non-normalised (Seed and Harder, 1990) and the other normalised (Olson and Stark, 2002) were adopted for modelling the ultimate lateral pressure from the liquefied soil on the pile, $p_{L\text{-}max}$. Key findings from a series of parametric analyses can be summarized as follows:

- Effects of shear strength normalisation of the liquefied soil could be significant for the pile response predicted by pseudo-static analysis. In the extreme case, the normalisation reduces the pile response from the ultimate level (failure) to the pre-cracking level (no damage).
- The magnitude of normalisation effects depends on the relative stiffness of the pile and the thickness of the non-liquefied crust at the ground surface.

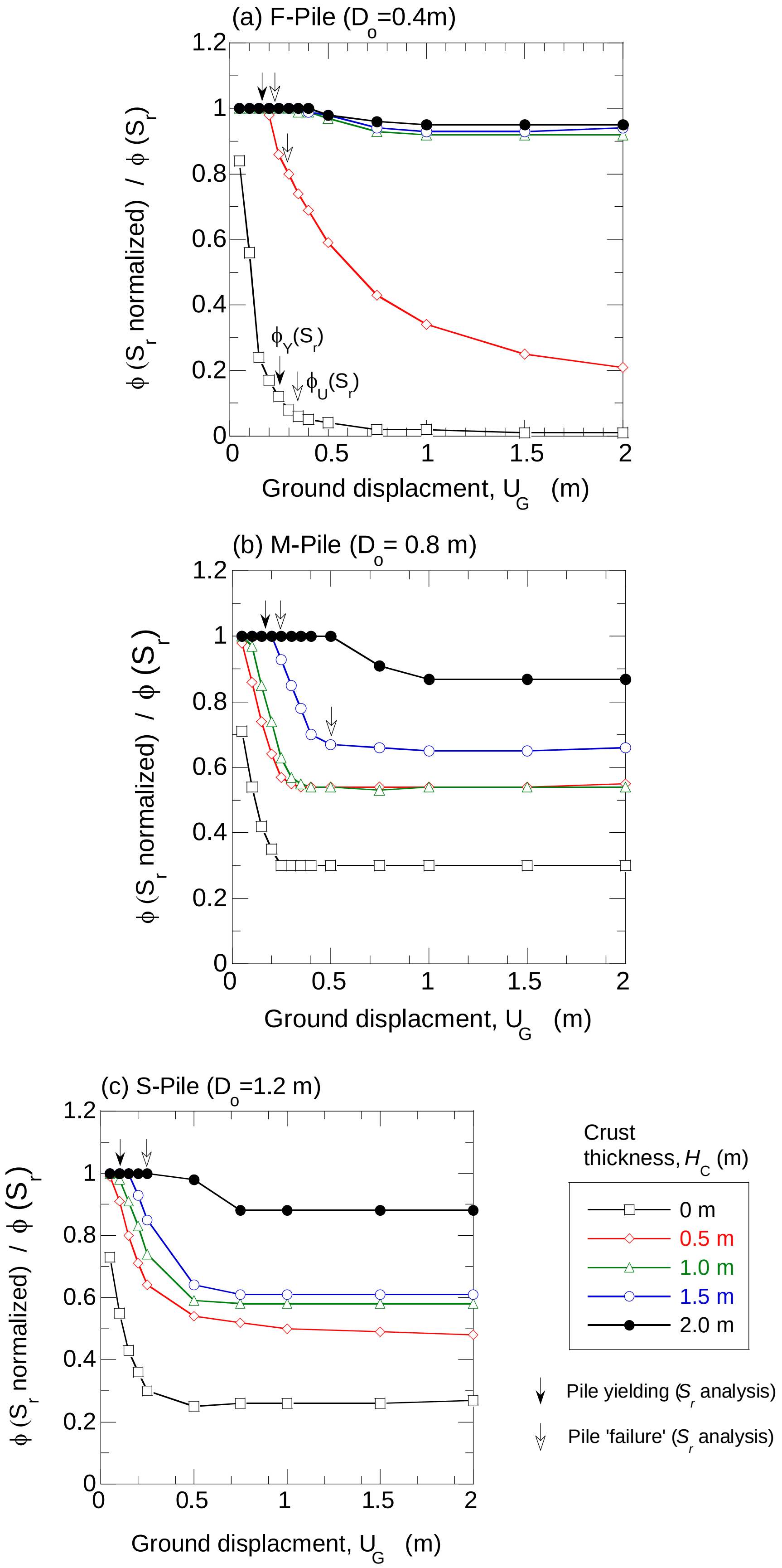

Figure 11. *Reduction in pile response due to strength normalisation, for different pile stiffness and thicknesses of the crust: (a) F-Pile; (b) M-Pile; (c) S-Pile*

- Effects of strength normalisation are largest in the absence of a crust, and decrease with the thickness of the crust. For the 10 m thick loose sand layer considered, the effects of soil strength normalisation were eliminated once the thickness of the crust exceeded 1.75 m.

It is important to recognize that the normalisation effects depend on the modelling of the ultimate load from the crust, which in this study was adopted to be 4.5 times the Rankine passive pressure. For other methods specifying smaller load from the crust, the effects of normalisation are expected to be greater than those presented herein. Similarly, one should note that the normalisation effects and derived threshold values for the thickness of the crust should be considered in the context of the adopted 10 m deposit of loose liquefiable soil.

Acknowledgement
The authors would like to acknowledge the financial support provided by the Earthquake Commission (EQC), New Zealand.

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CHAPTER 5: PSEUDO-STATIC MODELLING OF THE RESPONSE OF PILES IN LIQUEFYING SOIL: CYCLIC PHASE

Haskell, J., Cubrinovski, M. and Bradley B. (2009). UC Research Report (journal paper in preparation)

5.1 OVERVIEW OF THE PROBLEM

Strong earthquakes have been responsible for widespread damage to pile foundations in areas where extensive soil liquefaction and lateral spreading occurred. The interaction between the soil, pile(s), and superstructure (if present) is dynamic and intense, and many aspects of this interaction are not well understood. Many methods, of varying degrees of complexity and sophistication, have been developed for the design and analysis of piles in liquefying soils.

This study focuses exclusively on simplified, pseudo-static methods, intended for the preliminary design and analysis of pile foundations. Many such methods are included in seismic design codes, however no single method has been universally adopted, and all are burdened by significant uncertainties associated with soil liquefaction and seismic soil-pile interaction. The present lack of guidance for the consideration of these uncertainties precludes the consistent and reliable use of simplified, pseudo-static methods in practice.

Context of this Study

The study presented here addresses specifically the modelling and prediction of pile response during the 'cyclic' phase of loading, when the ground shaking is the most intense and the soil is liquefying. It forms part of a larger, ongoing programme of research, the ultimate goal of which is:

> *"to provide guidance for the consideration of uncertainties in the simplified analysis and design of pile foundations in liquefying and laterally spreading soils"*

The cyclic study comprises two distinct phases, the first considering the response of piles when subjected only to cyclic soil displacements, and the second considering the pile response when both cyclic soil displacements and superstructure inertial forces are present and acting simultaneously.

Guidance for the consideration of uncertainties during the post-cyclic 'lateral spreading' phase (when the strong shaking has ceased, but the liquefied soil has yet to regain its strength and may undergo large monotonic displacements) has been covered in a previous study (Haskell, 2008). The other phases of the larger research programme, still to be completed but not addressed here, include consideration of axial load effects on the pile response, extension from single-pile to pile-group analyses, and the application of the guidelines to case-histories and future projects.

Purpose of the Report

The primary purpose of this report is to provide a complete and comprehensive record of the cyclic phase analyses, the development and discussion of ideas as the study progressed, and the final outcomes of this phase of the research. Given the wider context of this study, much of the general background to the problem (such as a general overview of the lateral-load response of piles, or a review of the various methods for simplified pile analysis) is not covered here, but can be found in the lateral spreading phase report (Haskell, 2008).

5.2 RESEARCH DESIGN

The ultimate goal of users of simplified methods is to capture the essential features of the pile response and performance, which can be used as a basis for design decisions and a guide for further analyses. This requires a 'translation' of the complex soil-pile system that exists in reality into an equivalent simplified model that exhibits the same fundamental behaviour. Practically, this involves the selection (on the basis of incomplete information) of appropriate values or ranges of values for the various model parameters.

Phases of the Cyclic Study

This study comprises two phases. **The first phase** considers the pile response when the pile is subjected only to cyclic soil demands, in the form of lateral ground displacements. No inertial loads are applied to the pile in these analyses. In this phase, the sensitivity of the response to variations of critical soil parameters was considered for a wide range of soil-pile systems, so that trends in parametric sensitivity for the cyclic study can be compared directly with those already established in the lateral spreading study.

The second phase focuses on the effect of both the lateral ground displacement and the inertial force on the pile response. Here, changes to the 'reference response' caused by the simultaneous variation of the ground displacement and inertial force demands were studied, for all of the various soil-pile systems of the first phase. The reference response does not consider the influence of individual soil parameters, however for certain soil-pile-demand combinations the selection of soil parameter values may be very important. Additional analyses were thus conducted to identify those critical combinations, for which parametric sensitivity analyses were then undertaken.

Simplified Analysis Method

All analyses in this study were conducted using the simplified, pseudo-static method developed by Cubrinovski and others (Cubrinovski and Ishihara, 2004; Cubrinovski et al., 2006). The method is based on a beam-spring model of the soil-pile system that can accommodate tri-linear pile (beam element) behaviour, and bi-linear soil (spring) behaviour, as illustrated in Figure 1 for a three-layer soil profile. In this method, all loads are applied statically, the inertial superstructure demand being represented by a lateral point load at the pile head, and the pressure from the displacing liquefied soil being applied to the pile via static displacement of the springs of the liquefied (and crust) soil.

Soil Spring Calculations

The soil spring properties (i.e. strength, P-max, and stiffness, K) must be determined indirectly, on the basis of other soil properties, the results of in-situ soil tests, or via various established empirical relationships. The relationships and methods of calculation of the soil spring properties used for this study are identical to those of the lateral spreading study (Haskell, 2008), therefore only a brief summary is provided here. The values/ranges of values taken by certain input parameters do differ between the two studies however (as do the demands on the pile), reflecting the change in soil properties and behaviour as liquefaction develops. Figure 2 shows a typical bi-linear soil spring, and the flowchart of Figure 3 summarises the various parameters and steps involved in calculating the spring's strength, P-max, and stiffness, K.

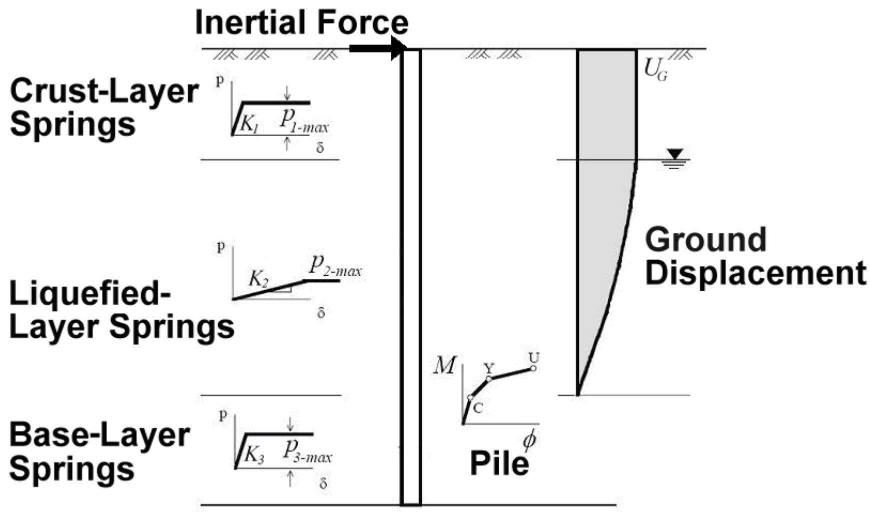

Figure 1. ***Beam-spring model of the soil-pile system showing tri-linear pile (beam element) properties and bi-linear soil (springs) properties***

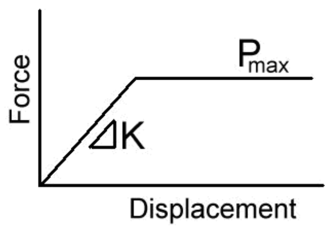

Figure 2. ***Typical bi-linear p-y curve for a soil spring, showing the strength and stiffness parameters P-max and K***

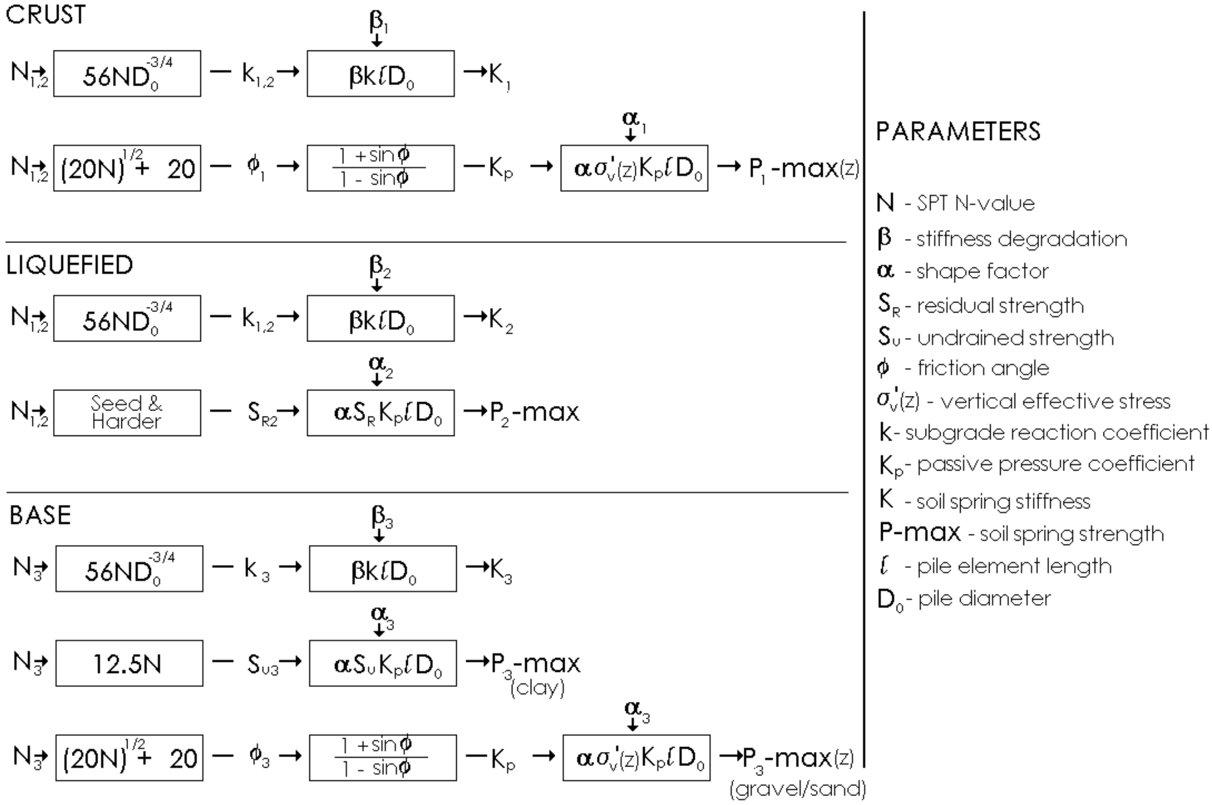

Figure 3. ***Flowchart showing the various soil properties and relationships that are used to determine the soil spring parameters***

Reference Model Approach

In reality, the possible combinations of soil types, layering, pile properties, and seismic demand are limitless. As such, the 'reference model' approach that was devised for the lateral spreading study is also adopted here. It involves the definition of a series of 'gross' soil-pile combinations (the four soil profiles of Figure 4 combined with the three reference piles of

Table 1) for which 'best-estimate' or 'reference' values are determined for all of the soil parameters. By varying, in turn, each of these parameters, their influence on the predicted pile response can be isolated and studied. These parametric analyses have been undertaken for a wide range of demand combinations (soil displacement, inertial force, and crust thickness) for each of the twelve reference models chosen for this study (which are identical to the twelve reference models of the lateral spreading study).

A key finding of the lateral spreading study was that only a few of the various input and intermediate parameters required for the calculation of the soil spring properties (i.e. the various parameters of Figure *3* have a critical influence on the predicted pile response. Therefore, only these critical soil parameters have been included in the cyclic parametric sensitivity studies. Table 2 summarises the reference value and range of variation for each of the critical soil properties (for all four reference soil profiles).

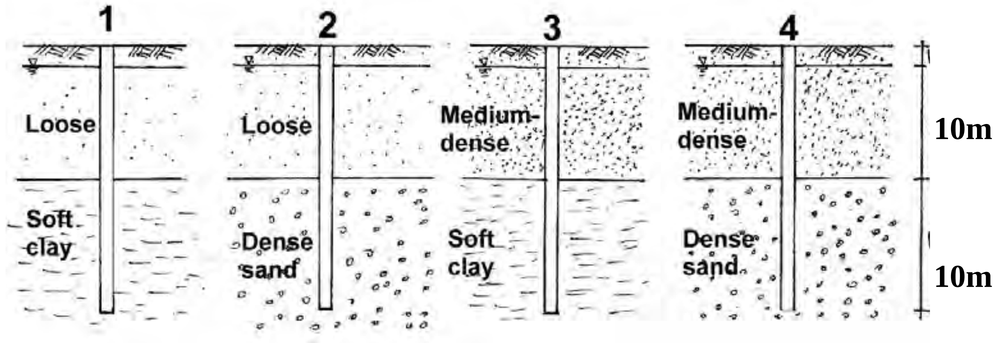

Figure 4. *Four reference soil profiles that, combined with the three reference pile, make up the twelve reference models of the cyclic study*

Table 1. Cracking, yielding, and ultimate moment (and curvature) capacities of the three reference piles

		S	M	F
D	[mm]	1200	800	400
M_c	[kN-m]	959	650	80.4
M_y	[kN-m]	1970	1240	126
M_u	[kN-m]	2470	1420	133
Φ_c	[m^{-1}]	0.00041	0.00109	0.00190
Φ_y	[m^{-1}]	0.00251	0.00609	0.01160
Φ_u	[m^{-1}]	0.01040	0.01169	0.02450

Table 2. Reference, minimum, and maximum values of the critical soil properties included in the parametric sensitivity studies

		P1 & P2 ($N_{1,2}$ = 5)			P3 & P4 ($N_{1,2}$ = 15)		
		Min	Ref	Max	Min	Ref	Max
β_2		0.02	0.05	0.1	0.02	0.05	0.1
α_2		1	3	6	1	3	6
S_{R2}	[kPa]	1	7	15	25	34	44

Cyclic Phase Demands

Uncertainties in the demand (both the cyclic soil displacement and the inertial superstructure force) are not treated in the same sense as uncertainties in the soil properties. Rather, the demand is varied over a wide range in order to explore any changes in the mechanism of soil-pile interaction and to capture the full range of pile response, from elastic pile behaviour through to pile failure.

For both phases of this cyclic study and all of the reference soil-pile systems, cyclic ground displacements of 0.1, 0.2, 0.3, and 0.4 m have been considered. For the second phase, which focuses on the influence of inertial loads, static point forces equivalent to 0.1g, 0.2g, 0.3g, 0.4g, and 0.5g were considered in combination with the four ground displacements, resulting in a total of 24 load combinations for each soil-pile system across the two phases. The axial capacities of the piles for each of the twelve reference models, along with the associated inertial forces are summarised in Table 3. In addition, the influence of the thickness of the non-liquefied surface or crust layer has been investigated by repeating the full suite of analyses for each soil-pile-demand combination for different thicknesses of crust (the total crust + liquefied soil layer thickness being kept constant, at 10m, for all analyses).

Table 3. Axial pile capacities and inertial force demands for the twelve reference Models

		S1 & S3	S2 & S4	M1 & M3	M2 & M4	F1 & F3	M2 & M4
P	[kN]	750	3000	500	1300	250	300
0.1g	[kN]	75	300	50	130	25	30
0.2g	[kN]	150	600	100	260	50	60
0.3g	[kN]	225	900	150	390	75	90
0.4g	[kN]	300	1200	200	520	100	120
0.5g	[kN]	375	1500	250	650	125	150

Summary

Table 4 below summarises all of the analyses that comprise the two phases of this study. In the table, ‘R’ indicates that an analysis was conducted using the ‘reference values for all parameters, while ‘P’ indicates that the full parametric suite of analyses, considering the individual influence of each of the critical soil parameters, was also undertaken.

Table 4. ***Summary of the reference and parametric analyses undertaken in this study for (a) the S-piles, (b) the M-piles, and (c) the F-piles***

(a)	S1				S2				S3				S4			
	F_I		No F_I		F_I		No F_I		F_I		No F_I		F_I		No F_I	
Crust [m]	R	P	R	P	R	P	R	P	R	P	R	P	R	P	R	P
0.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
0.5	✓		✓		✓		✓		✓		✓		✓		✓	
1.0	✓		✓		✓		✓		✓		✓		✓		✓	
1.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
2.5	✓		✓		✓		✓		✓		✓		✓		✓	

(b)	M1				M2				M3				M4			
	F_I		No F_I		F_I		No F_I		F_I		No F_I		F_I		No F_I	
Crust [m]	R	P	R	P	R	P	R	P	R	P	R	P	R	P	R	P
0.0	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
0.5	✓		✓		✓		✓		✓		✓		✓		✓	
1.0	✓		✓		✓		✓		✓		✓		✓		✓	
1.5	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

(c)	F1				F2				F3				F4			
	F_I		No F_I		F_I		No F_I		F_I		No F_I		F_I		No F_I	
Crust [m]	R	P	R	P	R	P	R	P	R	P	R	P	R	P	R	P
0.0	✓		✓	✓	✓		✓	✓	✓		✓	✓	✓		✓	✓
0.5	✓		✓		✓		✓		✓		✓		✓		✓	
1.0	✓		✓		✓		✓		✓		✓		✓		✓	
1.5	✓		✓	✓	✓		✓	✓	✓		✓	✓	✓		✓	✓

5.3 PHASE ONE: NO INERTIAL FORCE DEMAND

The primary purpose of these analyses was to determine whether or not the general sensitivity trends established for the critical soil parameters (β_2, α_2, and S_{R2}) for the lateral spreading phase of the pile response are also applicable to the cyclic phase of the response. These analyses represent something of an 'intermediate' scenario between the cyclic and lateral spreading phases, as the load on the pile is purely kinematic, just as for the lateral spreading analyses, but the magnitude of the ground displacement is much less and the liquefied soil is more stiff.

The analyses cover all twelve reference models, for profiles both with and without a non-liquefied crust. Two response parameters, the peak pile curvature and the pile head displacement, are used to evaluate the pile performance, as not only must failure of the foundation itself be avoided, but the safety and serviceability of the supported structure, which may be influenced by foundation displacements, must also be ensured. The full numerical results for all of the analyses of this first phase are provided in Appendix 2.

The results are presented here in the form of plots (
Figure *6* to Figure 8) relating the sensitivity of the peak pile curvature to the variation of each of the critical soil parameters (β_2, α_2, and S_{R2}) to the 'yield ratio' of the liquefied soil. The yield ratio (
Figure *5*) reflects the mechanism of soil-pile interaction, and is defined as the relative displacement between the soil and the pile at the ground surface (U_G – U_P) normalised by the yield displacement of the liquefied soil springs (Δy_2), calculated for the 'reference response'. The curvature sensitivity is defined as the difference between the peak pile curvature when the given parameter is varied from its upper bound value to its lower bound value (Φ_{UB} - Φ_{LB}), normalised by the reference value peak pile curvature (Φ_{ref}).

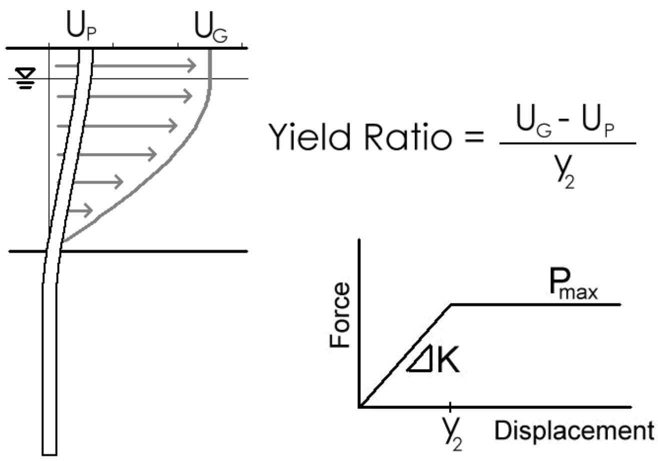

Figure 5. Diagram illustrating the physical meaning of the yield ratio

Sensitivity Trends

The sensitivity of the response to the critical parameters clearly depends on the relative soil-pile displacement (i.e. the yield ratio), and hence depends on the mechanism of soil-pile interaction. For yield ratios les than 1, which correspond to pile behaviour that is essentially flexible (i.e. the relative displacement between the soil and the pile is small), the sensitivity of the response to the stiffness degradation parameter, β_2, is at its peak. For larger yield ratios, which correspond to stiffer pile behaviour, the stiffness of the liquefied soil does not greatly affect the response and the value of β_2 is no longer important. This is because yield ratios greater than 1 indicate that much of the liquefied soil is yielding, and the resulting pressure on the pile is controlled by the strength of the bi-linear soil springs.

In contrast, the peak pile curvature is the least sensitive to the soil strength parameters α_2 and S_{R2} for low yield ratios, when the pile response is flexible. The predicted response becomes increasingly sensitive to any variation of these parameters the larger the yield ratio, until some constant level of sensitivity to α_2 and S_{R2} is reached, indicating significant yielding of the liquefied soil throughout most of its depth.

This ultimate ‘constant’ sensitivity is not a unique value, but rather depends on both the range of variation of the parameter (α_2 or S_{R2}), and the thickness of any non-liquefied crust layer, which represents an additional demand unaffected by the variation of the liquefied soil properties. The response is thus much more sensitive to α_2 and S_{R2} when there is no crust, as the pile is driven entirely by the liquefied soil.

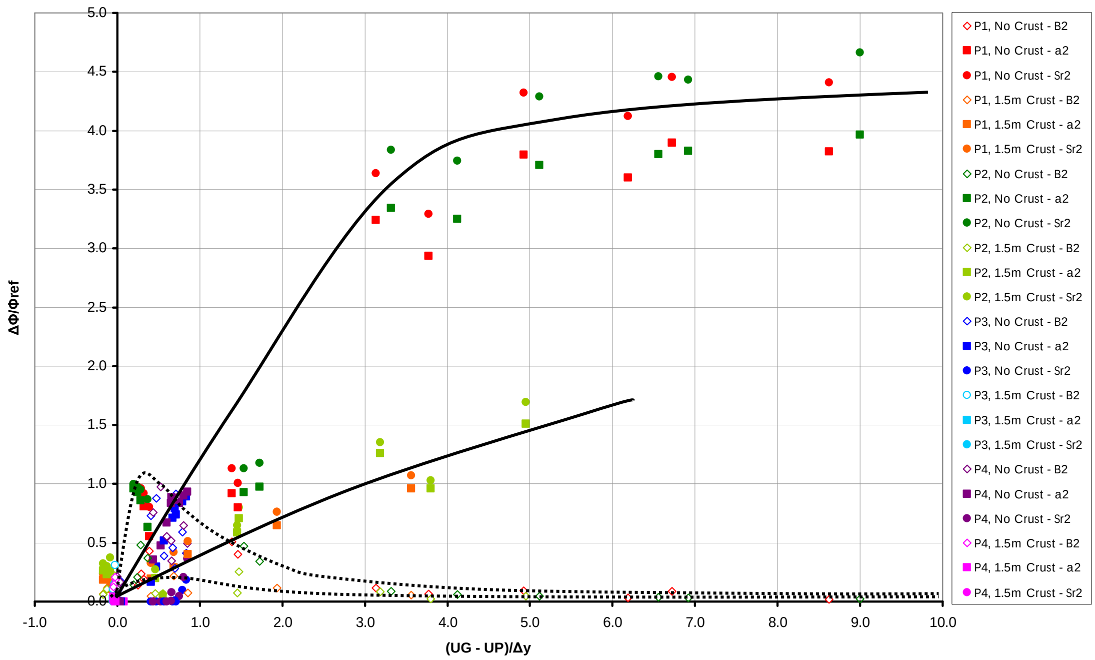

Figure 6. ***Parametric sensitivity results for liquefied soil parameters β_2, α_2, and S_{R2} for all piles for the cyclic phase, showing the increase in sensitivity to strength and the decrease in sensitivity to stiffness as more of the liquefied soil yields***

As expected, these sensitivity trends are exactly the same in nature to those of the lateral spreading phase. The use of the yield ratio to characterise the response means the different liquefied soil properties and lateral ground displacements between the cyclic and lateral spreading phases are allowed for, and the trends for each phase can be compared on common axes (Figure 7and Figure 8).

There is very good agreement between the curvature sensitivities for the two phases. The only appreciable difference, the somewhat greater sensitivity to β_2 for the lateral spreading phase, is likely a reflection of the greater range of variation of β_2 for the lateral spreading study as compared to the cyclic study (0.001 – 0.02, compared to 0.02 – 0.1).

5.4 PHASE TWO: KINEMATIC AND INERTIAL FORCE DEMAND

In this second phase of the cyclic study an additional demand, a static horizontal point force acting at the pile head, is combined with the lateral soil displacement already considered. Just as the static soil displacement represents the dynamic soil demand during strong ground motion, this static point load represents the dynamic force transmitted to the foundation from the superstructure above.

The modelling of this complex and dynamic behaviour presents significant challenges, in particular in the selection and combination of appropriate static demands. The peak cyclic soil displacement and superstructure inertial force are transient conditions, occurring only momentarily during the course of strong shaking. They may or may not occur at the same instant. It has been suggested that the phasing of kinematic and inertial demands varies, and depends primarily on the natural frequencies of the superstructure and the liquefied soil. At present, the load combination that corresponds to the critical pile response (i.e. the peak pile curvature and pile head displacement) cannot be predicted with any certainty.

The aim of this research is not to determine how best to define or calculate the critical pseudo-static model demand for a given scenario, but rather to investigate the influence of this modelling decision on the predicted pile response. For this reason, this study takes a very simple approach fro the treatment of demand combinations – each of the chosen soil displacements being combined, in turn, with each of the chosen inertial forces. No attempt has been made to correlate the inertial and kinematic demand, nor has an attempt been made to predict, a priori, the most critical load combination(s) for a given soil-pile system. Table 3 summarises the inertial loads for all of the soil-pile systems of this study.

Given the number of analyses required (24 demand combinations for each crust-soil-pile scenario), and the expectation that some of these demand combinations would predict unrealistic pile behaviour, the additional consideration of uncertainties in the liquefied soil properties (β_2, α_2, and S_{R2}) has been reserved for only those (realistic) scenarios where the liquefied soil properties are expected to be critical.

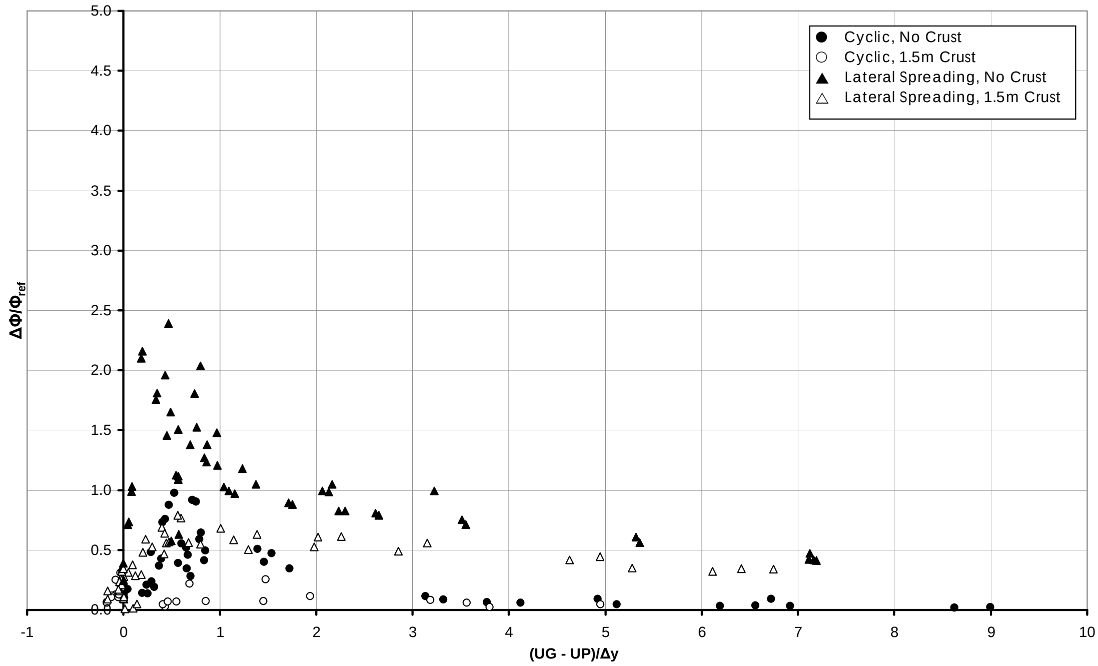

Figure 7. ***Sensitivity trends the liquefied soil stiffness parameter β_2, for both the cyclic and the lateral spreading phases, showing the close agreement between the two***

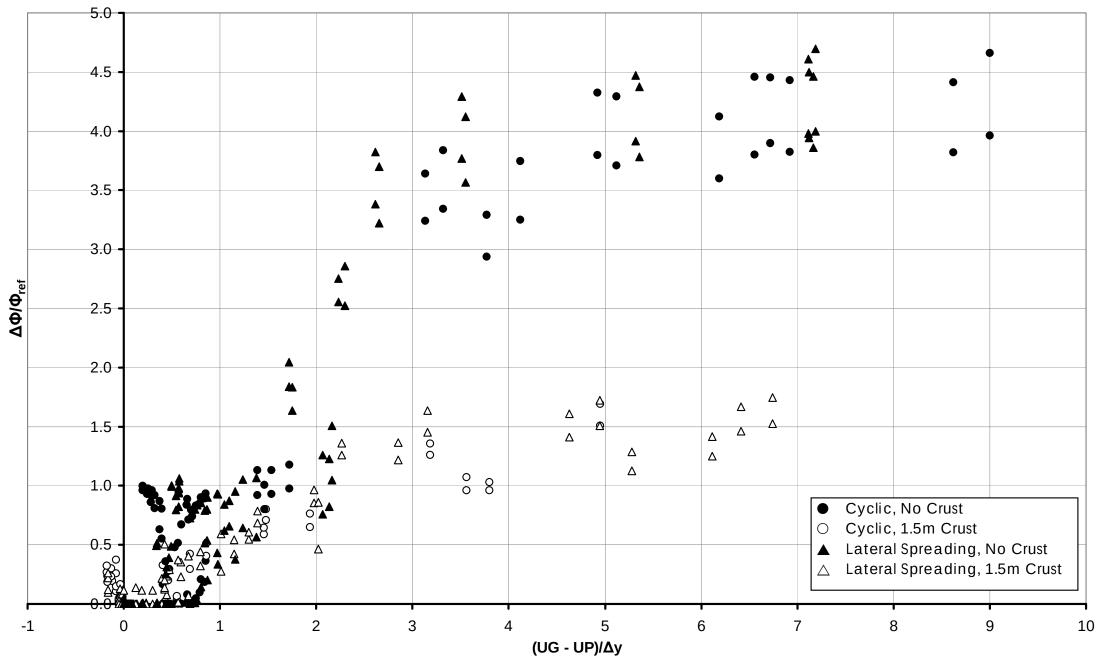

Figure 8. ***Sensitivity trends the liquefied soil stiffness parameter α_2 and S_{R2}, for Both the cyclic and the lateral spreading phases, showing the close agreement between the two and the influence of a non-liquefied crust layer***

In terms of the simplified pseudo-static analysis, the application of a point load at the pile head should have an effect somewhat similar to that of a displacing non-liquefied crust. To allow for a thorough comparison of the crust and inertial forces, the analyses presented here cover crusts of thickness 0, 0.5, 1.0, 1.5, and 2.5 m (2.5 m only for the 1200 mm diameter piles). Table 4 provides a summary of all of the analyses that comprise this second phase of the cyclic study, showing the inertial force-crust thickness combinations that have been considered.

Figure 9 compares the magnitude of these forces for the twelve reference soil-pile systems (noting that the 'crust force' refers to the maximum potential force from the crust, which requires a relative soil-pile displacement sufficient to fully yield all of the crust-layer soil springs).

Presentation and Interpretation of Results

The results for the first set of analyses (the reference response for each of the demand-crust-soil-pile combinations) are presented as plots (Appendix 1). Each of the twenty four plots corresponds to a single pile (S, M, or F), a single cyclic ground displacement (0.1, 0.2, 0.3, or 0.4 m), and a particular density of liquefied (and crust) soil (either loose or medium dense). The plots are presented four to a page, grouped by soil-pile combination, allowing the response of the pile to different displacement demands to be easily compared.
Figure *10* below is an example of one of these plots, and is used here to explain their general features.

Firstly, each 'dot' on a plot represents a separate analysis, and the 'shape' of the dot indicates the type of base layer for that particular analysis (where a rounded dot is used for reference profiles P1 and P3, which have a soft clay base layer, and a diamond-shaped dot is used for profiles P2 and P4, which have a dense gravel/sand base).

The position of each dot (i.e. the x and y axes) reflects the crust thickness/force and inertial force demand for that particular analysis. The y-axis indicates directly the applied inertial force in kN, while the x-axis indicates the maximum possible crust force (which is a function of the thickness of the crust for the given analysis). Depending on the response of the pile, this maximum crust force may or may not be fully mobilised (i.e. if the pile behaviour is relatively flexible and hence the relative soil-pile displacement is small, the true force from the crust will be less than this potential maximum). Furthermore, where the applied inertial load is particularly large, 'reverse' pile behaviour may occur, with the displacement of the pile being appreciably greater than that of the displacing soil. In this case the true resultant crust force may act in the opposite direction, having a restraining rather than a driving effect on the pile.

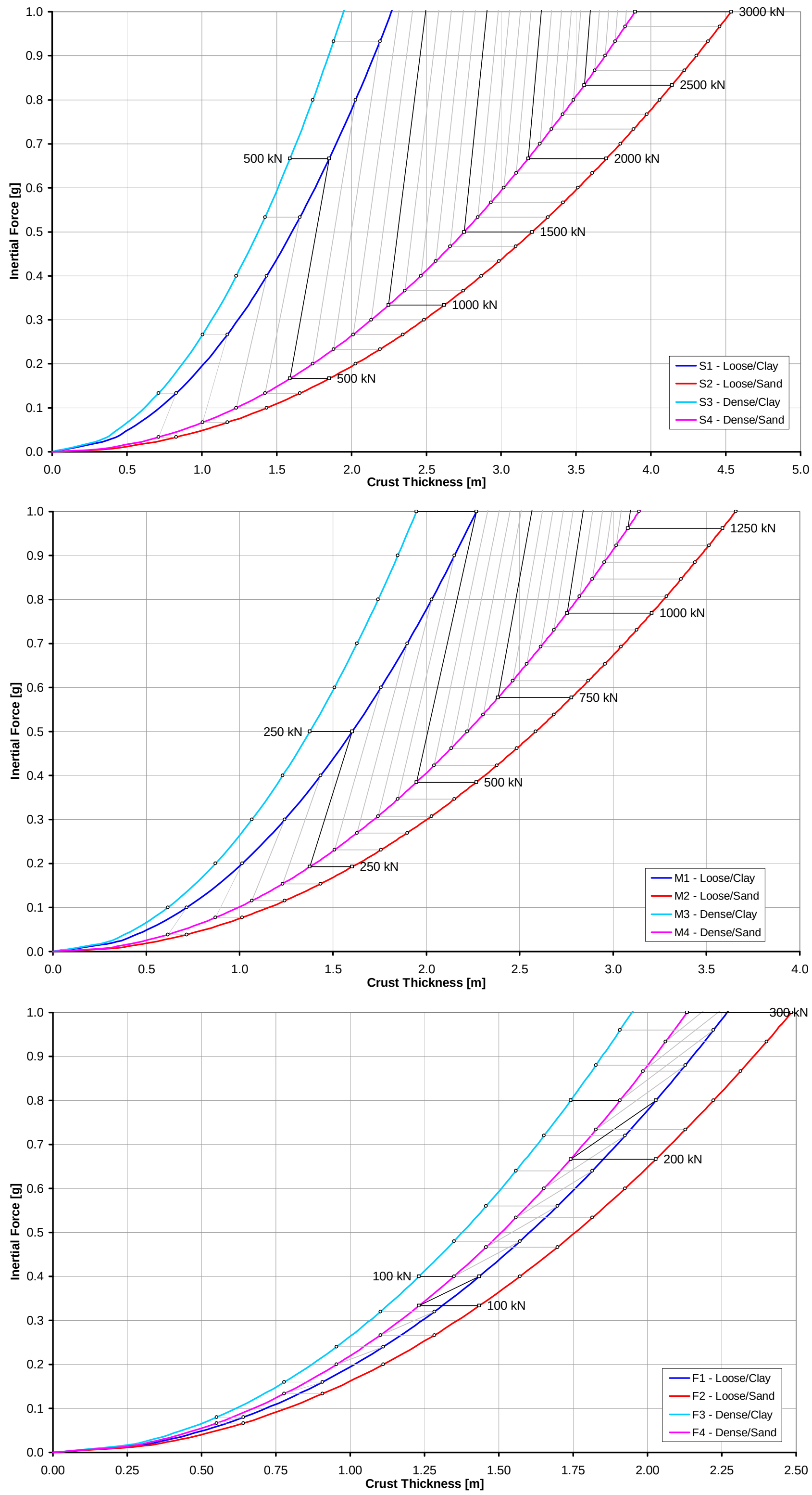

Figure 9. ***Equivalent inertial and (full yield) crust forces for the (a) S-piles, (b) M-piles, and (c) F-piles***

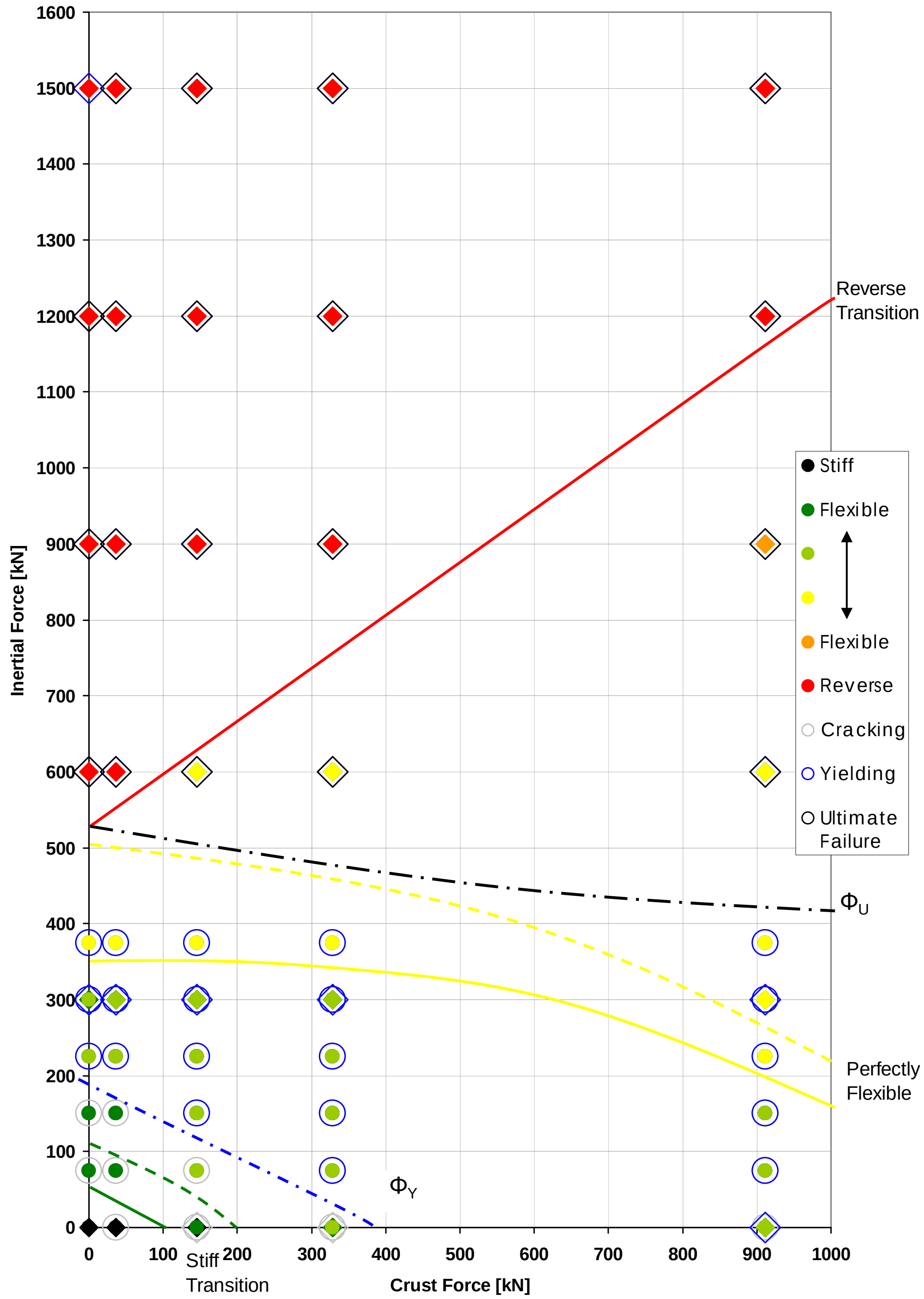

Figure 10. ***Plot showing the combined influence of a non-liquefied crust and an inertial superstructure force on the mechanism of response and severity of pile damage for reference model S1, where the ground displacement is 0.1m (presented here as an illustrative example)***

The colour of each dot indicates the yield ratio, the relative displacement between the soil and the pile at the ground surface normalised by the yield displacement of the liquefied soil (the soil yield displacement being the same for all of the analyses on a single plot), and hence the mechanism of interaction between the soil and the pile. The liquefied soil yield displacements for all models are summarised in Table 5.

Table 5. Yield displacement of the bi-linear liquefied soil springs (in m) for all soil-pile combinations

Liquefied Soil Density	S-Pile	M-Pile	F-Pile
Loose (P1 & P2)	0.0554	0.0409	0.0243
Medium-Dense (P3 & P4)	0.0892	0.0658	0.0391

The mechanism of response for a given analysis is broadly categorised based on the yield ratio as either:

Stiff		**yield ratio > 1**
Flexible	**-1 <**	**yield ratio < 1**
Reverse		**yield ratio < -1**

The latter of these, 'reverse' behaviour indicates that the pile is displacing significantly further than the liquefied soil, a mechanism of response that is not possible when no inertial load is applied, and has thus not been observed in lateral spreading study or phase one of this cyclic study. *Figure 11* shows the colours that correspond to the different mechanisms of response, and illustrates these mechanisms in terms of soil and pile displacement and mobilised soil spring force.

The plots also reveal the level of damage the pile sustained in each analysis, the colour of the border around each dot indicating the level of the peak pile curvature relative to the cracking, yielding, and ultimate curvatures of the pile (which are provided in Table 1).

No border		$\Phi_{peak} < \Phi_{crack}$
Grey	$\Phi_{crack} <$	$\Phi_{peak} < \Phi_{yield}$
Blue	$\Phi_{yield} <$	$\Phi_{peak} < \Phi_{ultimate}$
Black		$\Phi_{peak} > \Phi_{ultimate}$

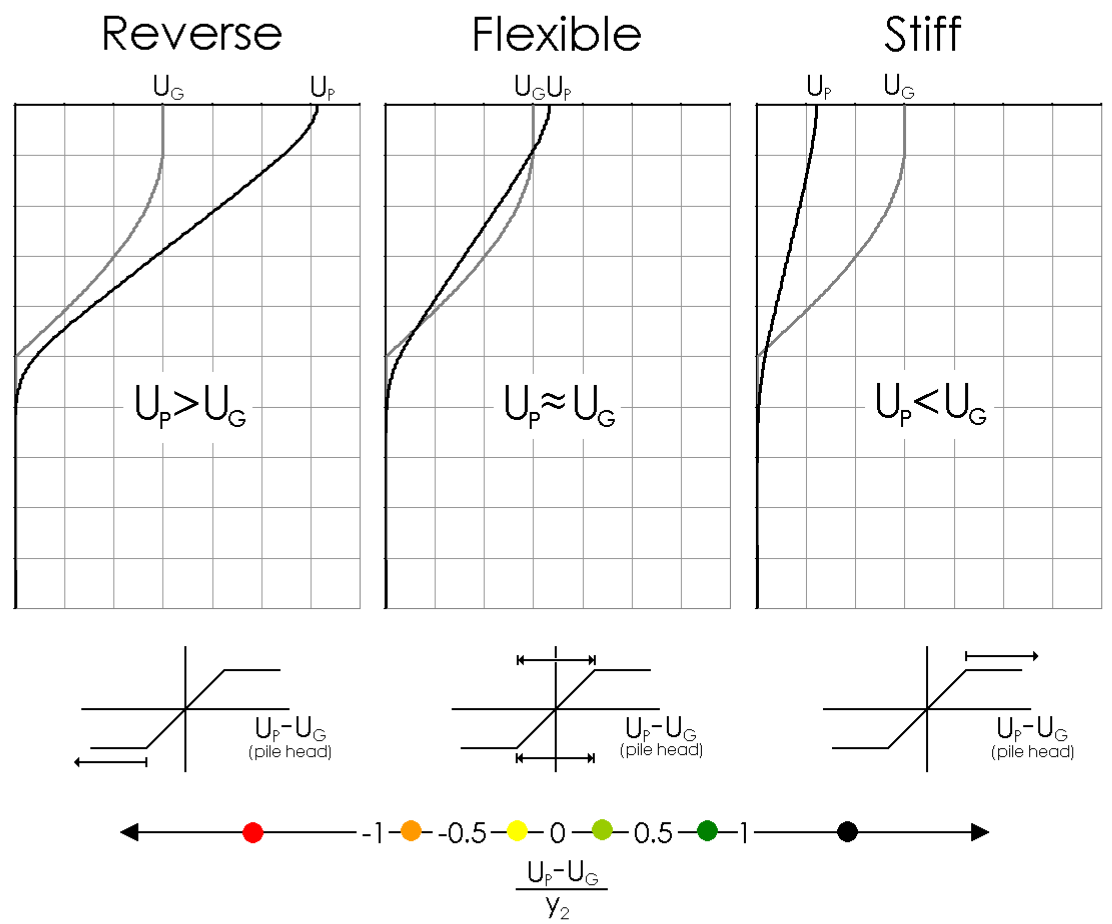

Figure 11. Definition of stiff, flexible, and reverse mechanisms of soil-pile interaction used for the cyclic inertial force study

Lastly, on each of the plots lines have been drawn to indicate general trends in the response and level of damage sustained by the pile as the inertial force and crust thickness are varied. They allow the influence of the crust and inertial forces to be easily compared across plots (i.e. for different ground displacements and soil-pile combinations). Specifically, there are lines to indicate the (approximate) boundaries between stiff and flexible behaviour, between flexible and reverse behaviour (referred to as the 'stiff transition' and 'reverse transition' lines, respectively), and a further line corresponding to perfectly flexible behaviour, when the soil and pile displacement at the ground surface are exactly the same. Similarly, lines are used to indicate the crust-inertial force combinations that cause the first onset of pile yielding and the first onset of ultimate pile failure for the given ground displacement and soil-pile combination.

Trends in Pile Damage

The peak pile curvature (and hence the damage sustained by the pile) is very closely related to the displacement of the pile. This is reflected by the trends in the yield and ultimate failure lines for each soil-pile combination for different ground displacement demands. Where the crust and/or liquefied soil are capable of causing flexible pile behaviour, these damage lines vary in position as the ground displacement is increased, as the pile is essentially moving together with the displacing soil. Therefore, for a given soil-pile-crust combination, the inertial force required to induce pile yield or ultimate failure reduces as the ground displacement is increased and the damage line shifts closer towards the origin.

However, where the pile behaviour is stiff and the pile has sufficient strength and stiffness to resist the ultimate pressure form the displacing soil, the positions of the damage lines are independent of the ground displacement (changes to the ground displacement have no effect on the mobilised soil pressure, as the relative displacement between the soil and the pile is sufficient to yield most of the crust and liquefied soil). For a given crust thickness, the inertial force required to induce a certain pile head displacement and peak pile curvature is thus independent of the level of cyclic ground displacement.

Furthermore, once stiff behaviour is achieved, the crust and inertial forces are essentially additive, and the force at the pile head required to induce a particular level of damage can be made up of any combination of crust and inertial force. This 'direct combination' of crust and inertial forces is, in fact, more widely applicable, however as the pile behaviour becomes more flexible the full crust force is unlikely to be mobilised. This shifting of demand from the crust to the inertial force is discussed in more detail in previous sections, however its effect on the damage lines is essentially to 'flatten' them, the inertial force having a greater influence on the level of damage than the crust thickness.

The inertial force required to cause a particular level of damage for a given soil-pile-crust-ground displacement combination does depend somewhat on the base soil, in particular the discontinuity of soil stiffness at the liquefied-base soil interface. Where the base soil is stiffer (i.e. the gravel-base profiles P2 and P4), the discontinuity is greater and a lower inertial force is generally required to reach a certain level of damage.

In general, for a given soil-pile combination and ground displacement, the inertial force required to induce a certain level of damage decreases as the thickness of the crust is increased, reflecting the similar effect each has on the pile response. In some cases however, the required inertial force initially reduces the thicker the crust, but then as the crust thickness is further increased, the required inertial force increases. This is caused by a change in the location of the peak pile curvature, from the pile head to lower within the non-liquefied crust, causing the peak curvature to decrease slightly in magnitude. This is illustrated in Figure 12. This trend is 'transient', appearing only where the pile behaviour is relatively flexible and the ground displacement (and hence the pile displacement) is of a magnitude that corresponds to the yield or ultimate curvature of the pile.

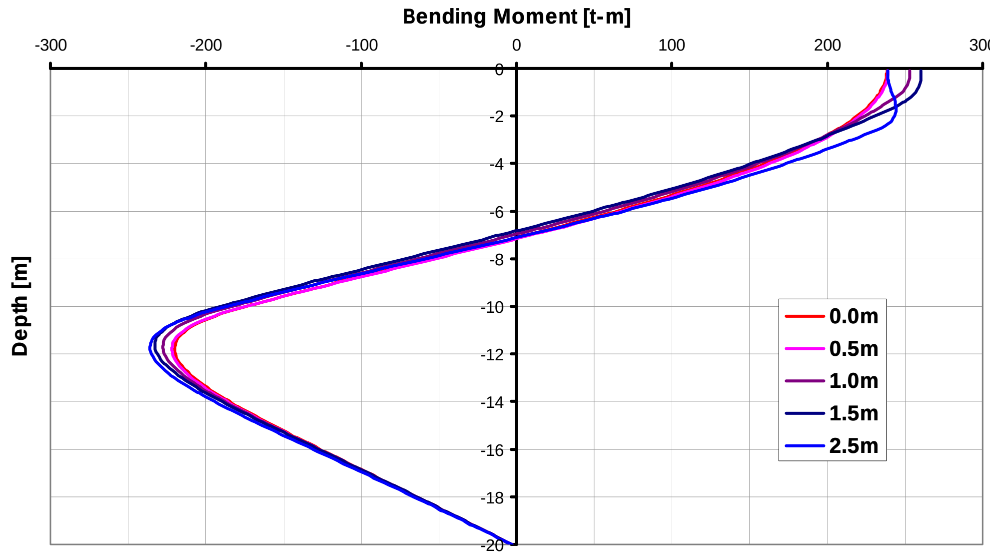

Figure 12. Pile bending moment distributions for crusts of different thickness, showing the shift in location of the peak moment as the crust thickness is increased

Trends in the Mechanism of Soil-Pile Interaction

The shear number of uncertain parameters or 'dimensions' of the pseudo-static method that need to be considered in the prediction of pile performance makes it very difficult to develop a universal or 'big picture' description of every aspect of the soil-pile model, for all possible scenarios of pile loading. The challenge is not so much to develop absolute, quantitative formulae or relationships to describe very precisely the influence of individual parameters on the pile response (such relationships are inevitably burdened by so many 'conditions' that they are of no practical use. Rather, the challenge is finding the best position from which to develop a qualitative, physical understanding of the soil-pile system as a whole.

It is clear from the results of the previous phases that the mechanism of interaction between the soil and pile is the key to interpreting and understanding the behaviour of all aspects of the system. On this basis, the effect of the inertial force on the pile response is examined via the stiff and reverse 'transition' lines of the figures of Appendix 1, and the following trends have been identified and explained:

- For a given pile, ground displacement, and inertial load, the nature of the pile response (i.e. stiff, flexible, or reverse) does not depend on the properties of the base layer. This means that the profiles P1 and P2 can be considered together (as can the profiles P3 and P4), despite the significant difference in base layer properties for the two profiles. In contrast, models having a different density liquefied (and crust) soil do not exhibit similar response for comparable inertial force and crust demands. The profiles having a loose liquefied soil layer (P1 and P2) are thus presented separately from those having medium-dense liquefied soil (profiles P3 and P4).
- Across all models, the 'stiff transition' and 'perfect flexibility' lines have a downward gradient, indicating that the thicker the non-liquefied crust, the

smaller the inertial force required to cause flexible and perfectly flexible behaviour. Conversely, the ‘reverse transition’ line generally has an upward gradient, which reflects the change in role of the crust force as reverse loading occurs, with the crust now resisting the pile displacement and the action of the inertial force. The thicker the crust, the greater the potential resisting force.

- For those models where stiff behaviour can be achieved, the stiff transition exhibits a clear trend as the ground displacement is increased, shifting away from the origin until the relative displacement between the soil and the pile is sufficient to mobilise a significant proportion of the maximum pressure from the displacing soil (i.e. the liquefied and crust layers). The position of this line is then independent of the ground displacement, as further increases in ground displacement will not affect the load on the pile (hence the inertial force required to cause a transition to flexible behaviour for a given crust thickness, which controls the position of the line, becomes fixed). At this point, the inertial and crust force intercepts to the stiff transition line are more or less equal as stiff behaviour implies the full crust force is developed and the actions of the two forces are essentially equivalent.
- It is clear that the roles of the crust and inertial forces are relatively straightforward when the pile response is stiff, or when the inertial force is so large that significant ‘reverse’ pile behaviour (and reverse yielding of the crust) occurs. However, when the pile response is flexible, and the relative soil-pile displacement is small, the interaction between the crust force and the inertial load is more complex. The zone of flexible behaviour on the plots (as it is defined here) is the entire region between the stiff and reverse transition lines.

  This zone of flexible behaviour is significantly ‘larger’ for those profiles where the liquefied and crust soil is medium-dense, as opposed to loose (i.e. for the medium-dense soil, a greater proportion of crust thickness and inertial force combinations result in flexible pile behaviour). In fact, for the three piles considered here, the pressure from the medium-dense liquefied soil alone is sufficient to cause flexible pile behaviour, even when there is no non-liquefied crust or inertial load at the pile head. Similarly, the inertial force required for reverse response is greater, the more dense the liquefied soil. Hence, the zone of flexible pile response broadens as the density of the liquefied soil is increased.
- The flexible zone is essentially a ‘buffering’ region in which changes to the inertial force are offset or mitigated by changes to the resultant crust and/or liquefied soil force. The width of this region (i.e. the difference in inertial force between the stiff and reverse transition lines, for a given crust thickness) reflects the capacity of the liquefied and crust soil to absorb or accommodate changes to the inertial force, with minimal effect on the pile displacement and peak pile curvature. It follows that this buffered zone widens as the crust thickness is increased and is everywhere wider where the liquefied soil is more dense. *Figure 13* illustrates the nature of the interaction between the inertial force and the resultant driving soil force for stiff, flexible, and reverse pile behaviour, while *Figure 14* illustrates the change in crust force at the pile head as the crust thickness is increased for a pile that is initially stiff, but becomes more flexible.

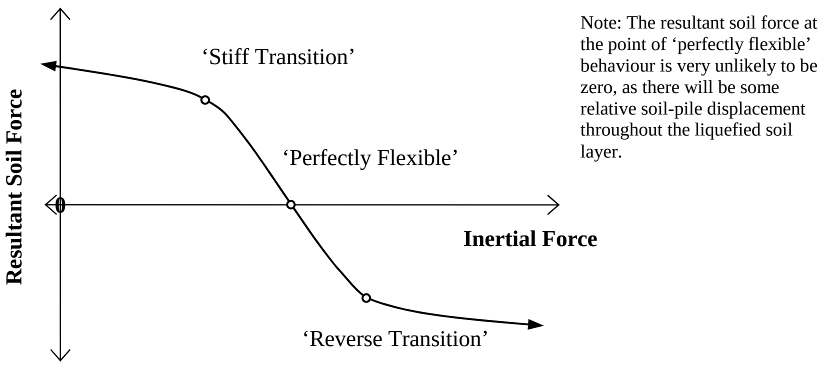

Figure 13. Conceptual illustration of the interaction between the inertial force and the resultant force from the displacing soil as the mechanism of soil-pile interaction changes from stiff to reverse

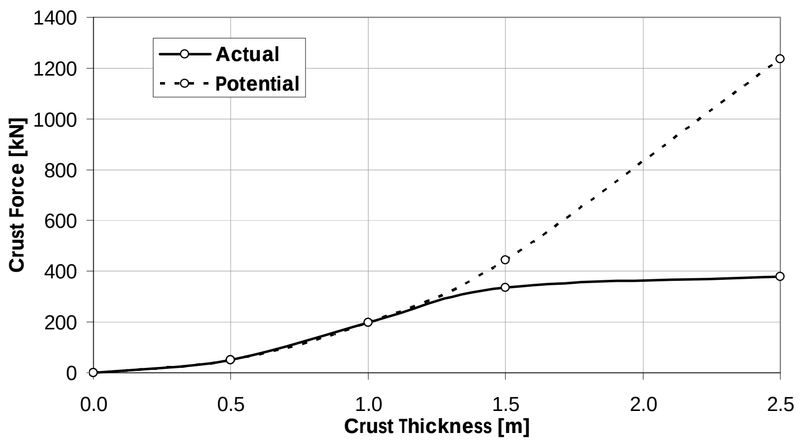

Figure 14. Actual versus potential force from the non-liquefied crust, as a function of crust thickness (for S3)

Influence of the Inertial Force on Parametric Sensitivities

The influence of the inertial force on the parametric sensitivity trends of Phase One for the cyclic study has also been considered. A preliminary suite of soil parameter sensitivity analyses, conducted in exactly the same manner as those of Phase One, has been undertaken for all of the S-pile models, for soil profiles both with and without a non-liquefied crust, and all 24 inertial force-ground displacement demand combinations. The results of these analyses were used to identify the soil-pile-demand combinations sensitive to the values of individual liquefied soil parameters. Combinations that were obviously unrealistic (i.e. where the predicted pile displacement was significantly larger than the cyclic ground displacement), along with those predicting an unacceptable level of pile damage (well in excess of the ultimate curvature capacity of the pile) were immediately excluded.

Table 6 summarises the (relatively few) S-pile scenarios where the influence of individual liquefied soil parameters is considered to be important.

Unsurprisingly, these are the scenarios that correspond to stiff or relatively stiff pile behaviour. Extending this trend to the M and F piles, the following soil-pile demand combinations (Table 7) have also been identified as being potentially sensitive to variation of the liquefied soil parameters (note that no F models are included, as the F-pile is not stiff enough to resist the demands considered in this study).

The parametric sensitivities for the chosen models, plotted as curvature sensitivity versus yield ratio, are presented in *Figure 15* to *Figure 18*, where each colour and symbol corresponds to a different soil-pile combination and inertial force demand.

Table 6. S-pile scenarios for which liquefied parameters are important

	S1	S2	S3	S4
No Crust	✓	$F_I \leq$ 0.2g	$F_I \leq$ 0.3g	×
1.5m Crust	×	$F_I \leq$ 0.2g	×	×

Table 7. M and F-Pile scenarios fro which liquefied soil parameters are expected to be important

	M1	M2	M3	M4	F1	F2	F3	F4
No Crust	✓	$F_I \leq$ 0.2g	$F_I \leq$ 0.3g	×	×	×	×	×
1.5m Crust	×	$F_I \leq$ 0.2g	×	×	×	×	×	×

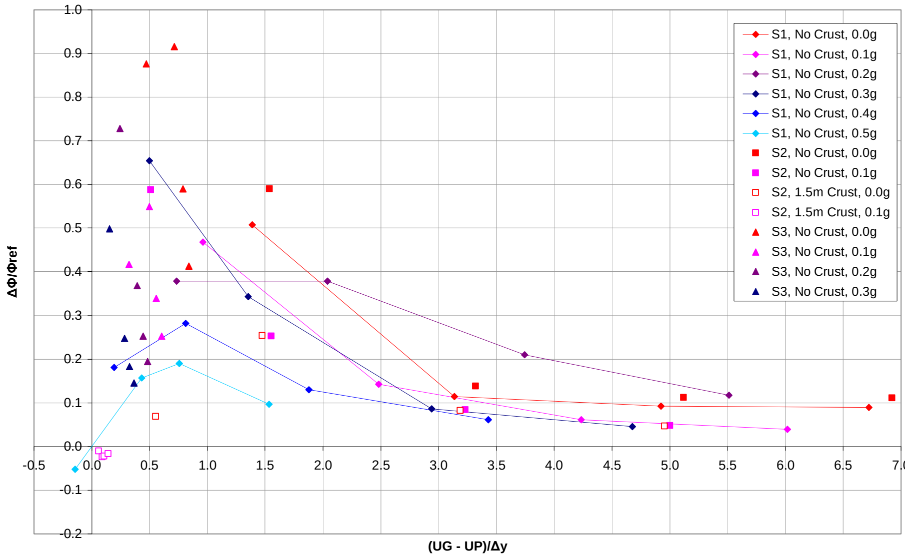

Figure 15. ***Inertial force influence on the S-pile sensitivity to the stiffness degradation parameter*** β_2

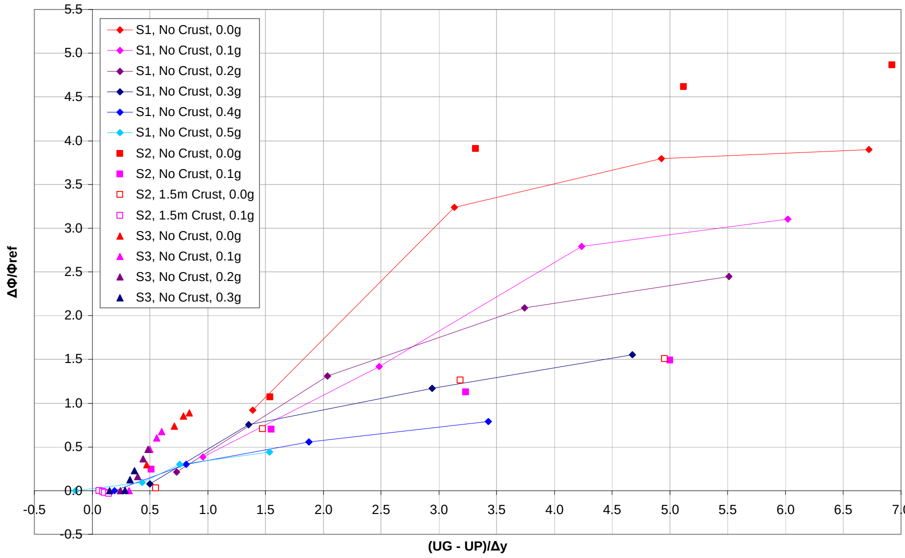

Figure 16. ***Inertial force influence on the S-pile sensitivity to the stiffness degradation parameter*** $\alpha 2$

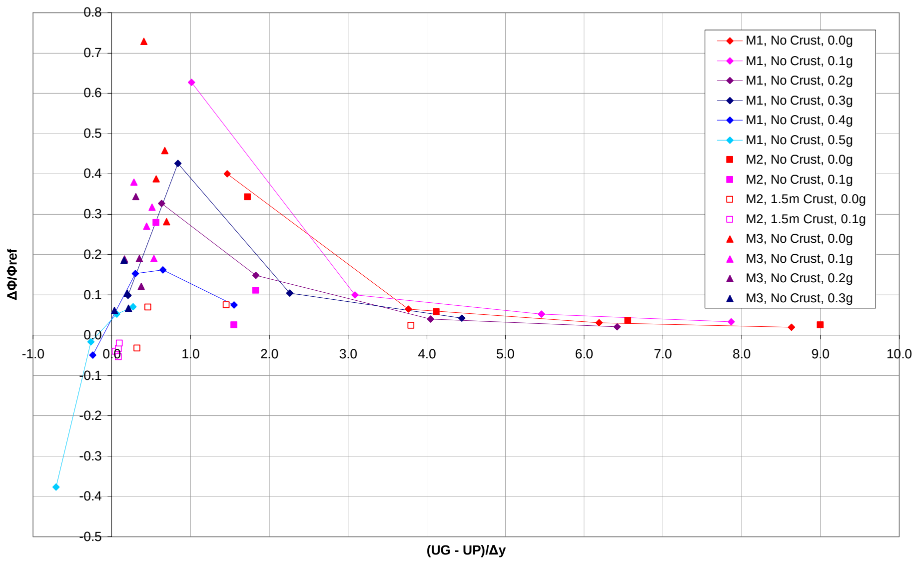

Figure 17. ***Inertial force influence on the M-pile sensitivity to the stiffness degradation parameter β2***

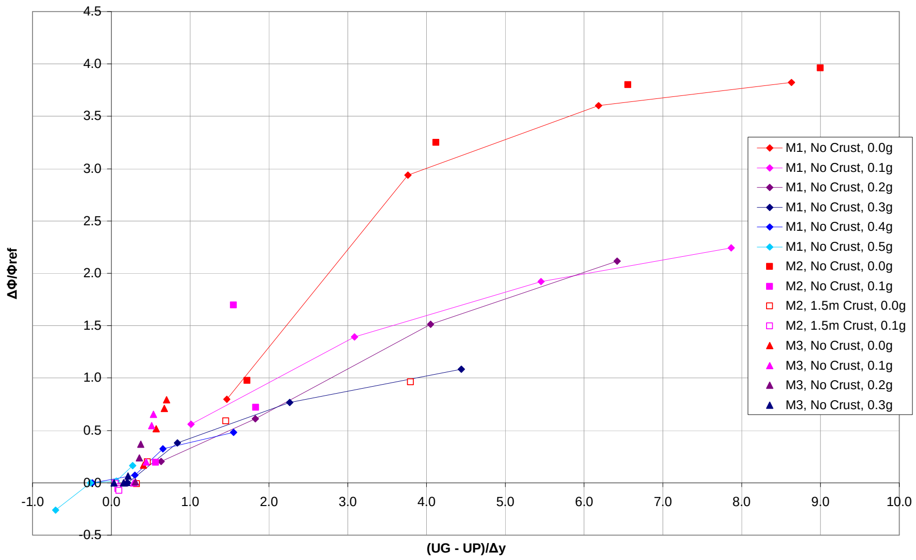

Figure 18. ***Inertial force influence on the M-pile sensitivity to the stiffness degradation parameter α2***

For a given soil-pile combination, the application and increase of an inertial force have two key effects:

1. The addition of an inertial force increases the pile displacement for the given model, reducing the relative soil-pile displacement, making the response more flexible and decreasing the yield ratio.
2. In general, for a particular yield ratio, the sensitivity to β_2, α_2, and S_{R2} is lower, the larger the inertial force. This effect is analogous to that of a non-liquefied crust on the sensitivity of the response to the liquefied soil parameters. Indeed, where the yield ratio is greater than 1, equivalent crust and inertial forces have an identical effect on liquefied soil sensitivity.

5.5 SUMMARY AND CONCLUSIONS

This study considers the simplified, pseudo-static modelling of single piles in liquefying soils during the cyclic phase of earthquake loading, when the shaking is the most intense. The piles are subjected to kinematic loads arising from the relative displacement between the pile and the liquefying soil, along with inertial forces at the pile head due to the motion of the superstructure above. This study focuses on the influence of the selection of model parameter values on the predicted pile response, an understanding of which is essential for the consistent and reliable use of pseudo-static methods in practice.

The results of the Phase One parametric study, which considered the influence of critical soil parameters on the predicted response when no inertial demand is present, confirm the fundamental link between the mechanism of soil-pile interaction and the relative importance of different soil properties. As for the previous lateral spreading study, the relative displacement between the pile and the liquefied soil, as compared to the ‘yield’ displacement of the soil, serves as an index of this response, ‘collapsing’ the sensitivity results for a wide range of soil-pile demand combinations to well-defined, general trends. When the pile behaviour is flexible, the stiffness of the liquefied soil is the most important, whereas when the pile behaviour is stiff, the liquefied soil strength controls the kinematic demand on the pile. The presence of a non-liquefied crust reduces the sensitivity of the response to the liquefied soil parameters. Where an inertial force is present, but the pile response is still relatively stiff (and thus sensitive to individual soil parameters), this force has the same influence on the parametric sensitivities as a non-liquefied crust.

The introduction of an inertial force at the pile head in addition to the kinematic soil demand adds another dimension to this already complex problem. For a given scenario, this force may potentially change the fundamental mechanism of soil-pile interaction, increase the severity of the damage suffered by the pile, and alter the influence other model parameters have on the predicted pile response.

This interaction between the inertial force and all aspects of the model behaviour can be understood qualitatively using the ‘buffering capacity’ concept developed in this study. The ‘buffering capacity’ of a given soil-pile model is essentially “its ability (or capacity) to absorb, accommodate, or compensate for changes to the input model parameters without appreciably changing the overall pile response". It is a function of the mechanism of soil-pile interaction and the potential liquefied (and crust) force that can be mobilised.

Specifically, where the pile response is flexible, (i.e. the relative displacement between the soil and pile is small), the soil-pile system is able to absorb changes to the inertial force (or any other parameter) via small adjustments to the mobilised soil pressure, which require only small movements of the pile. The larger the potential soil pressure (i.e. the thicker the non-liquefied crust and/or the more dense the liquefied soil), the greater the magnitude of variations that can be absorbed. In contrast, where the pile behaviour is stiff (or where reverse response occurs), a significant portion of the potential soil pressure has already been mobilised. Changes to the inertial force demand can thus significantly alter the pile displacement (and damage) without greatly affecting the mobilised soil pressure. The overall behaviour of the soil-pile system is perhaps best summarised as follows. The pile 'wants' to be flexible. Its behaviour will be different only where:

1. The pile is so stiff and strong, and the driving soil and inertial force are so weak that stiff pile behaviour occurs, or
2. The inertial force is so large that it overwhelms the soil and pile, resulting in reverse response.

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CHAPTER 6: EVALUATION OF SEISMIC PERFORMANCE OF GEOTECHNICAL STRUCTURES

Cubrinovski, M. and Bradley B. (2009).Theme Lecture, International Conference on Performance-Based Design in Earthquake Geotechnical Engineering, IS-Tokyo2009, 15-17 June, Tsukuba, Japan, 121-136.

Abstract

Three different approaches for assessment of seismic performance of earth structures and soil-structure systems 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. Even though the methods nominally have the same objective, it is shown that they focus on different aspects in the assessment and provide alternative performance measures. Key features of the approaches and their specific contribution in the assessment of geotechnical structures are illustrated using a case study.

6.1 INTRODUCTION

Methods for assessment of the seismic performance of earth structures and soil-structure systems have evolved significantly over the past couple of decades. This involves improvement of both practical design-oriented approaches and advanced numerical procedures for a rigorous dynamic analysis. In parallel with the improved understanding of the physical phenomena and overall computational capability, new design concepts have been also developed. In particular, the Performance Based Earthquake Engineering (PBEE) concept has emerged. In broad terms, this general framework implies engineering evaluation and design of structures whose seismic performance meets the objectives of the modern society. In engineering terms, PBEE specifically requires evaluation of deformations and associated damage to structures in seismic events. Thus, the key objective in the evaluation of the seismic performance is to assess the level of damage and this in turn requires detailed evaluation of the seismic response of earth structures and soil-structure systems.

Clearly this is an onerous task since the stress-strain behaviour of soils under earthquake loading is very complex involving effects of excess pore-water pressures and significant nonlinearity. The ground response usually involves other complex features such as:

- Modification of the ground motion (earthquake excitation for engineering structures)
- Large ground deformation and excessive permanent ground displacements
- A significant loss of strength, instability and ground failure, and
- Soil-structure interaction effects.

The assessment of seismic performance of geotechnical structures is further complicated by uncertainties and unknowns in the seismic analysis. Particularly significant are the uncertainties associated with the characterization of deformational behaviour of soils and ground motion itself. Namely, the commonly encountered lack of geotechnical data for adequate characterization of the soil profile, in-situ soil conditions and stress-strain behaviour of soils results in uncertainties in the modelling and prediction of ground deformation. Even more pronounced are the uncertainties regarding the ground motion (earthquake excitation to be used in the analysis) arising from the inability to predict the actual ground motion that will occur at the site in the future.

The above uncertainties affect key elements in the analysis, the input load (ground motion or earthquake load) and constitutive model (stress-strain curve or load-deformation relationship). Clearly, the output of the analysis will be adversely affected by these uncertainties and would therefore require careful interpretation. One may argue that, strictly speaking, a prediction of the seismic response is not possible under these circumstances; instead, the aim should be an assessment of the seismic performance. This argument is not in the realm of semantics, but it rather implies difference in philosophy. It alludes to the importance of the process and engineering interpretation rather than the outcome alone, which is in agreement with the traditional role that engineering judgement has played in geotechnical engineering.

In this paper, three approaches for assessment of seismic performance are applied to a case study of a bridge on pile foundations. Conventional methods of seismic analysis are used in the assessment and comparatively examined. Key features in the implementation of the methods, their advantages and disadvantages are discussed. It is demonstrated that the examined approaches focus on different aspects and make different contribution in the assessment.

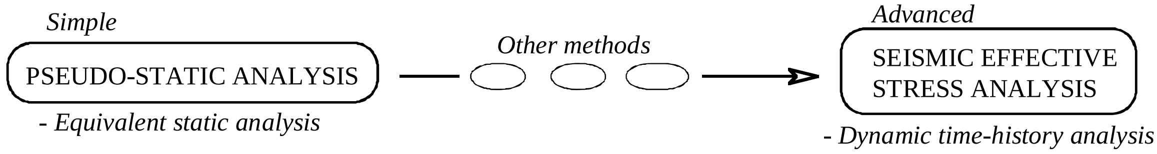

Figure 1. Methods for seismic analysis of earth structures and soil-structure systems

6.2 METHODS FOR ASSESSMENT OF SEISMIC PERFORMANCE

Analysis methods

There are various approaches for seismic analysis of earth structures and soil-structure systems ranging from relatively simple approximate methods to very rigorous but complex analysis procedures. These approaches differ significantly in the theoretical basis, models they use, required geotechnical data and overall complexity. The simplest methods are based on the pseudo-static approach in which an equivalent static analysis is used to estimate the dynamic response induced by the earthquake. The pseudo-static analysis is based on routine computations and use of relatively simple models, and hence is easy to implement in practice. For this reason, it is the commonly adopted approach in seismic design codes. On the other hand, the most rigorous analysis procedure currently available for evaluation of the seismic response of soil deposits and earth structures is the seismic effective stress analysis. This analysis permits detailed evaluation of the seismic response while considering the complex effects of excess pore water pressures and highly nonlinear behaviour of soils in a rigorous dynamic (time history) analysis. Despite its complexity, the seismic effective stress analysis is now frequently used in geotechnical practice for assessment of the seismic performance of important structures. As indicated in Figure 1, a large number of alternative analysis methods are available in the range between these two benchmark approaches.

Deterministic versus probabilistic approaches

Generally speaking, the seismic response can be evaluated either deterministically or probabilistically. Figure 2 illustrates the three approaches scrutinized in this study in this regard: (i) Deterministic approach (DA) in which a single scenario is considered; in this case, only one analysis is conducted and respectively a single response of the system is computed; (ii) Deterministic approach (DA_P) in which a series of analyses are conducted in a parametric manner in order to account for the uncertainties and unknowns in the analysis; as indicated in Figure 2, this approach results in a range of different responses for the analyzed system; (iii) Probabilistic approach (PA) in which “all possible” earthquake scenarios are considered for the site in question; this approach also results in a range of different responses for the system and, in addition, provides an estimate for the likelihood of each response.

The key difference between these three approaches is in the treatment of the uncertainties. The deterministic approach with a single scenario (DA) effectively ignores the uncertainties in the analysis while the probabilistic approach (PA) offers the most rigorous treatment of uncertainties and quantifies their effects on the computed seismic response.

Adopted approaches

This paper examines three approaches for assessment of the seismic performance in the context outlined above as follows:

(1) Pseudo-static analysis within a deterministic approach incorporating parametric evaluation (DA_p)
(2) Seismic effective stress analysis using a single scenario (DA)

(3) Probabilistic approach based on the so-called PEER framework (Cornell and Krawinkler, 2000) using the seismic effective stress analysis as a computational method (PA)

These assessment approaches can be applied to various earth structures and soil-structure systems, but here they are applied to the assessment of seismic performance of pile foundations in liquefiable soils.

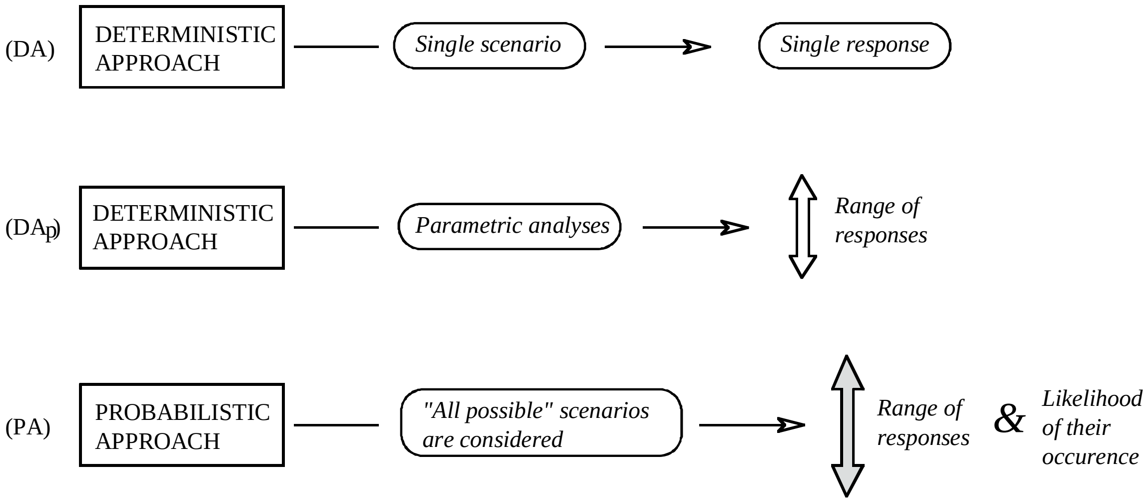

Figure 2. General approaches for assessment of seismic performance of geotechnical structures

Case study

The Fitzgerald Avenue Bridge over the Avon River in Christchurch, New Zealand, will be used as a case study. It is a small-span twin-bridge that has been identified as an important lifeline for post-disaster emergency services. Hence, the bridge has to remain operational in the event of a strong earthquake. To this goal, a structural retrofit has been considered involving widening of the bridge and strengthening of the foundation with new large diameter piles. A cross section at the mid span of one of the bridges is shown in Figure 3 where both existing piles and new piles are shown.

Figure 4 depicts the SPT blow count and soil profile at the northeast corner of the bridge. This soil profile was adopted in the pseudo-static 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 low fines content predominantly in the range between 3% and 15% by weight. Detailed SPT and CPT investigations revealed a large spatial variability of the penetration resistance at the site. Hence, a rigorous investigation of the seismic response of the bridge and its foundation would require consideration of 3-D effects and spatial variability of soils. These complexities are beyond the scope of this paper, however, and rather a simplified scenario will be considered herein with the principal objective being to examine the response of the pile foundation shown in Figure 3. Here, we will focus on the cyclic response of the foundation during the intense ground shaking; effects of lateral spreading are beyond the scope of this study.

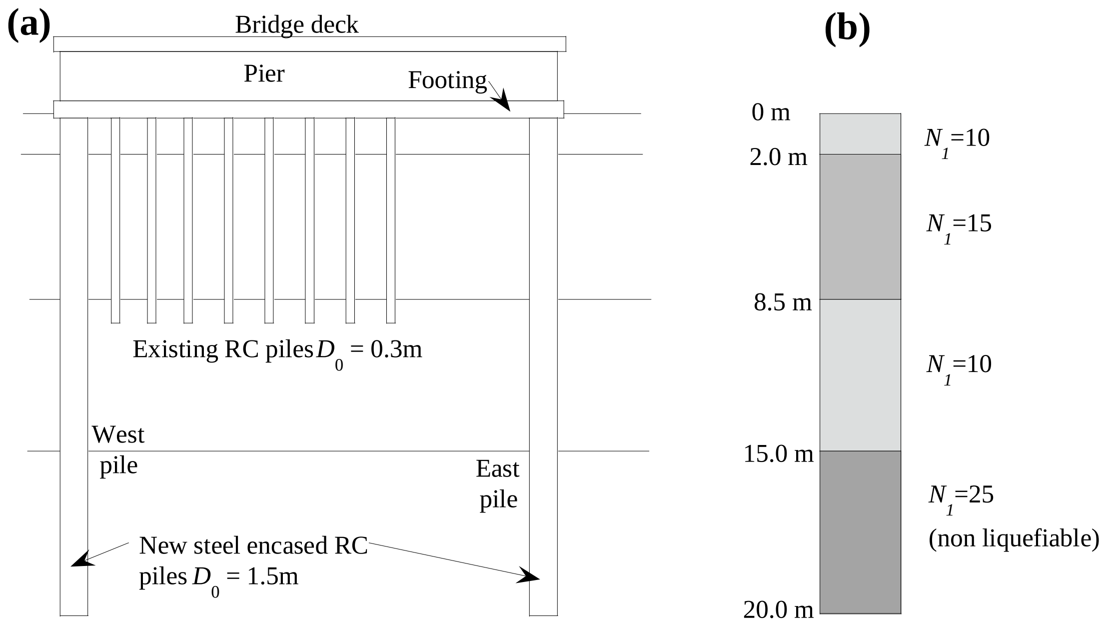

Figure 3. Central pier of the bridge: (a) cross section; (b) simplified soil profile used in seismic effective stress analyses (Bowen and Cubrinovski, 2008)

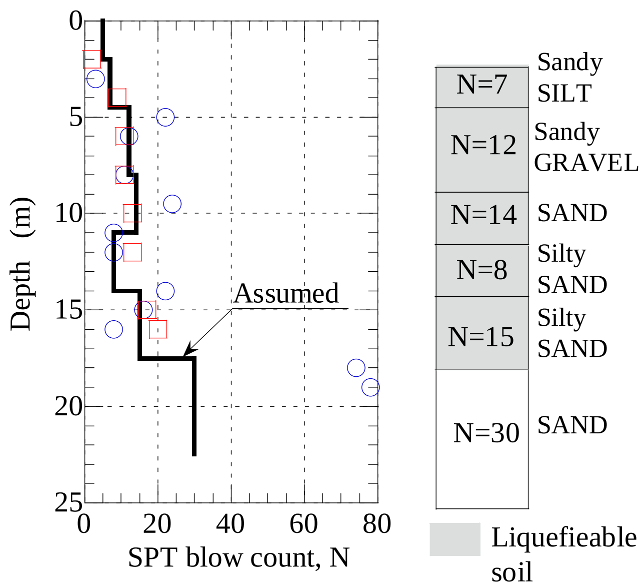

Figure 4. SPT blow count and soil profile at the north-east abutment

6.3 PSEUDO-STATIC ANALYSIS

Objectives

As a practical approach, the pseudo-static analysis should be relatively simple, based on conventional geotechnical data and applicable without requiring significant computational resources. In addition, in order to satisfy the PBEE objectives in the seismic performance assessment, the pseudo-static analysis of piles should:

- Capture the relevant deformational mechanism for piles in liquefying soils
- Permit estimation of the inelastic response and damage to piles, and
- Address the uncertainties associated with seismic behaviour of piles in liquefying soils.

Not all available methods for simplified analysis satisfy these requirements. In particular, in the current practice the treatment of uncertainties in the simplified analysis is often inadequate; commonly, the uncertainties are either ignored or poorly addressed in the analysis. In what follows, a recently developed method for pseudo-static analysis of piles in liquefying soils (Cubrinovski and Ishihara, 2004; Cubrinovski et al., 2009) is used to assess the seismic performance of the new piles of Fitzgerald Bridge. Key features of the simplified analysis and effects of uncertainties on the pile response are discussed.

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 a 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 5. The model can easily incorporate a stratified soil profile (multi-layer deposit) with different thickness of liquefied layers and a crust of non-liquefiable soil at the ground surface. Since one of the key requirements of the analysis is to estimate the inelastic deformation and damage to the pile, in the proposed model simple but non-linear load-deformation relationships are adopted for the soil-pile system. The soil is represented by bilinear springs in which degraded stiffness and strength of the soil are used to account for effects of nonlinear behaviour and liquefaction. The pile is modelled using a series of beam elements with a tri-linear moment-curvature relationship. Parameters of the model are illustrated in Figure 5 for a typical three-layer configuration in which a liquefied layer is sandwiched between a surface layer and a base layer of non-liquefiable soils. All model parameters are based on conventional geotechnical data (SPT blow count) and concepts (subgrade reaction coefficient, Rankine passive pressure). In the model, two equivalent static loads are applied to the pile: a lateral force at the pile-head (F) representing the inertial load due to vibration of the superstructure, and a horizontal ground displacement (U_G) applied at the free end of the soil springs (Fig. 5b) representing the kinematic load on the pile due to lateral movement of the free field soils.

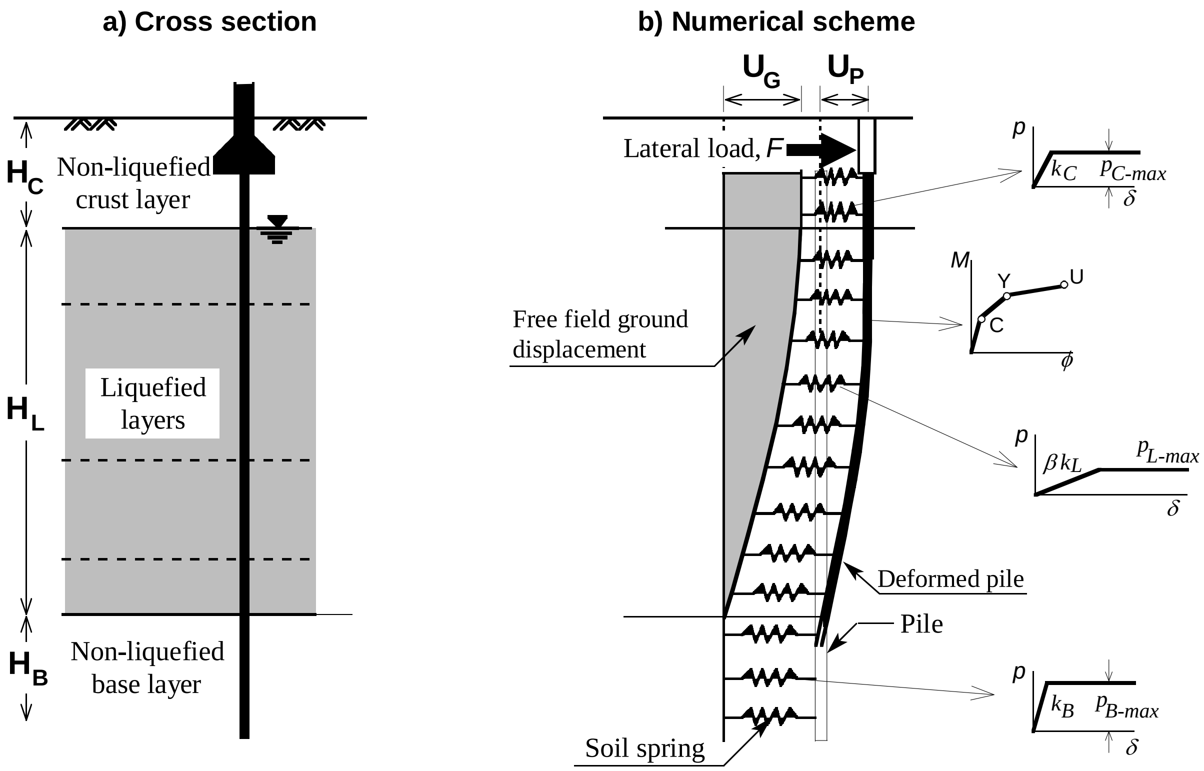

Figure 5. Beam-spring model for pseudo-static analysis of piles (model parameters and characterization of nonlinear behaviour)

Uncertainties in the parameters of the model

The pseudo-static analysis aims at estimating the maximum response of the pile under the assumption that dynamic loads can be idealized as static actions. Since behaviour of piles in liquefying soils is extremely complex involving very large and rapid changes in soil stiffness, strength and lateral loads on the pile, the key question in the implementation of the pseudo-static analysis is how to select appropriate values for the soil stiffness, strength and lateral loads on the pile for the equivalent static analysis. In other words, what are the appropriate values for β, $p_{L\text{-}max}$, U_G and F in the model shown in Figure 5? The following discussion illustrates that this choice is not straightforward and that all these parameters may vary within a wide range of values.

In the adopted model, effects of liquefaction on stiffness of the soil are taken into account through the degradation parameter β. Observations from full-size experiments and back-calculations from case histories indicate that for cyclic liquefaction (excluding lateral spreading), β typically takes values in the 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 $p_{L\text{-}max}$ in the model. The ultimate lateral pressure $p_{L\text{-}max}$ can be approximated using the residual strength of liquefied soils (S_r) as $p_{L\text{-}max} = \alpha_L S_r$. There are significant uncertainties regarding both α_L and S_r values. The latter is illustrated by the scatter of the data in the empirical correlation between the residual strength of liquefied soils and normalized SPT blow count $(N_1)_{60cs}$ (Seed and Harder, 1991) shown in Figure 6. For example, for a normalized equivalent-sand blow count of $(N_1)_{60cs}$ = 10, the residual strength varies approximately between 5 kPa and 25 kPa.

The selection of appropriate equivalent static loads is probably the most difficult task in the pseudo-static analysis. This is because both input loads in the pseudo-static analysis (U_G and F) are in effect estimates for the seismic responses of the free field ground and soil-pile-structure system respectively. The magnitude of lateral ground displacement U_G can be estimated using simple empirical models based on SPT charts such as that proposed by Tokimatsu and Asaka (1998). Using this method, a value of U_G = 0.36 m was estimated for the maximum cyclic ground displacement at Fitzgerald Bridge site. Note that since U_G is an estimate for the free field response at the site, it is reasonable to expect a considerable variation in the value of U_G around the above estimate based on an empirical model.

As mentioned earlier, the objective of the pseudo-static analysis is to estimate the peak response of the pile that will occur during an earthquake. The peak loads on the pile due to ground movement and vibration of the superstructure do not necessarily occur at the same time, and hence, there is no clear and simple strategy how to combine these loads in a static analysis. 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 whereas I_c is a factor that depends on the period of the earthquake motion and practically provides a rule for combining the kinematic (U_G) and inertial (F) loads on the pile. Again, a wide range of values have been suggested for this parameter: I_c = 0.4, 0.6 and 0.8 for a short, medium and long period ground motions respectively (Boulanger et al., 2007).

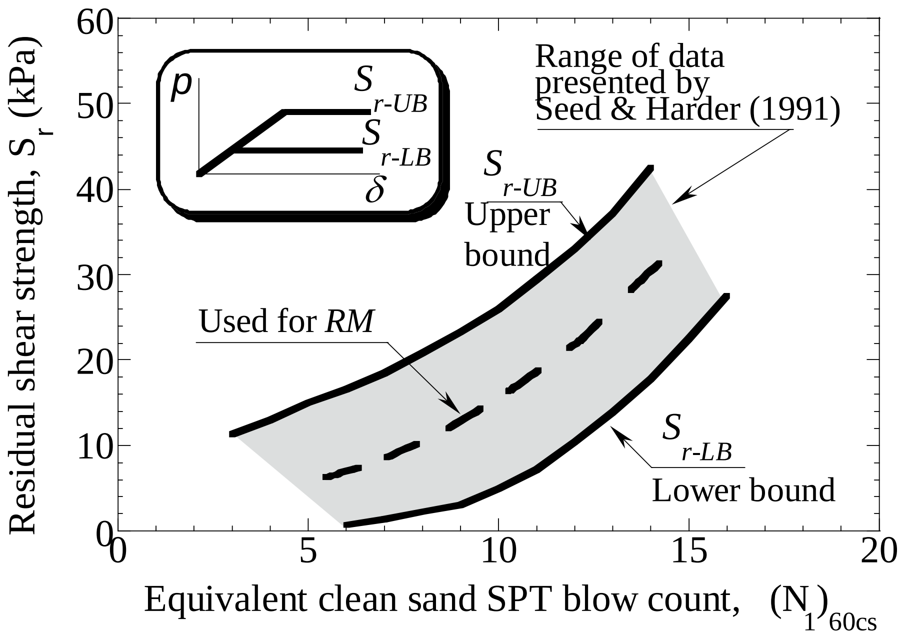

Figure 6. Residual shear strength of liquefied sandy soils (after Seed and Harder, 1991)

Computed response for a reference model (RM)

Based on the procedures outlined above, a so-called *reference model* (RM) was defined for the pile foundation of Fitzgerald Bridge. RM is a single pile model for the new piles (1.5m in diameter) in which a 'mid range' values were adopted for the parameters of the model, as summarized in Table 1. Here, the S_r values of 14 and 36 were derived using the broken line in Figure 6 and normalized blow counts of $(N_1)_{60cs}$ = 10 and 15 respectively, for the liquefiable layers. The pile was subjected to a free field ground displacement with a peak value at the ground surface of U_G = 0.36m, indicated in Figure 7a, and a lateral load at the pile head corresponding to a peak ground acceleration of a_{max} = 0.4g and an inertial coefficient of I_c = 0.6. The computed pile displacement and bending moment for the reference model (RM) are shown with solid lines in Figures 7a and 7b respectively. A pile head displacement of 0.21m and a peak bending moment at the pile head of 9.6 MN-m were computed. The bending moment exceeded the yield level both at the pile head and at the interface between the liquefied layer and underlying base layer.

Effects of uncertainties on the pile response

To examine the effects of uncertainties associated with the liquefied soil and lateral loads on the pile, parametric analyses were carried out in which the above parameters were varied within the relevant range of values listed in Table 1. For example, an analysis was conducted in which RM values were used for all parameters except for the stiffness degradation (β) and residual strength (S_r) of the liquefied soil, for which instead the lower bound or minimum values of β = 1/50, S_r = 6 kPa (N_1 = 10) and S_r = 24 kPa (N_1 = 15) were used. Similarly, another analysis was conducted in which the upper bound or maximum values of β = 1/10, S_r = 22 kPa (N_1 = 10) and S_r = 48 kPa (N_1 = 15) were used in conjunction with the RM values for all other parameters. Results of these two analyses are shown in Figure 7 indicating significant effects of the spring properties for the liquefied soil on the pile response.

Figure 8 shows results from a similar pair of analyses in which the value for the applied ground displacement was either decreased (U_G = 0.29m) or increased (U_G = 0.43m) for 20% with respect to the RM displacement of 0.36m. Again, a large difference in the pile response is seen resulting from a relatively small variation in the ground displacement applied to the pile.

Table 1. Characteristic values of model parameters

Parameter	**RM**	Range of values		
		LB*		UB**
β	**1/20**	1/50	-	1/10
S_r $(N_1=10)$ (kPa)	**14**	6	-	22
S_r $(N_1=15)$ (kPa)	**36**	24	-	48
I_c-	**0.6**	0.4	-	0.8
U_G (m)	**0.36**	0.29	-	0.43

* Lower Bound (minimum value)
** Upper Bound (maximum value)

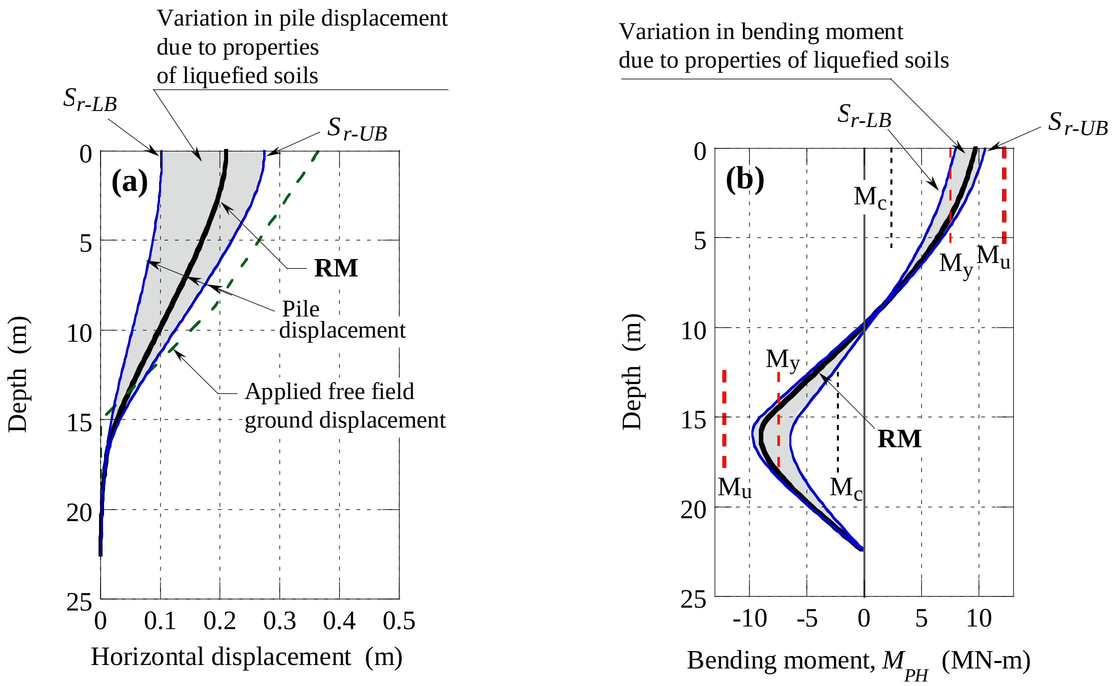

Figure 7. Effects of properties of liquefied soils on the pile response computed in the pseudo-static analysis: (a) pile displacements; (b) bending moments

Results of the parametric analyses are summarized in Table 2 and are depicted in tornado charts for the pile head displacement and bending moment (at the pile head) respectively in Figures 9a and 9b. The response of the reference model (RM) is also indicated in these plots for comparison purpose. The results clearly indicate that the pile response is significantly affected by the adopted values for stiffness and strength of the liquefied soil, and to a lesser extent by the adopted values for loads, U_G and F (due to variation of I_c between 0.4 and 0.8). Note that the size of these effects will change with the properties of the soil-pile system (especially with the stiffness of the pile relative to that of the soil), degree of yielding in the soil and pile, and the size of lateral loads from a non-liquefied crust at the ground surface.

Table 2. Results of parametric analyses

Model	Pile response	
	U_{PH}*	M_{PH}**
RM with $S_{r\text{-}LB}$ and $\beta_{2\text{-}LB}$	0.10	7.8
RM with U_G = 0.29m	0.16	8.9
RM with $I_{s\text{-}LB}$ = 0.4	0.18	8.9
RM	**0.21**	**9.5**
RM with U_G = 0.43m	0.25	9.9
RM with $I_{s\text{-}UB}$ = 0.8	0.23	10.0
RM with $S_{r\text{-}UB}$ and $\beta_{2\text{-}UB}$	0.27	10.3

* Pile-head displacement
** Bending moment at pile head

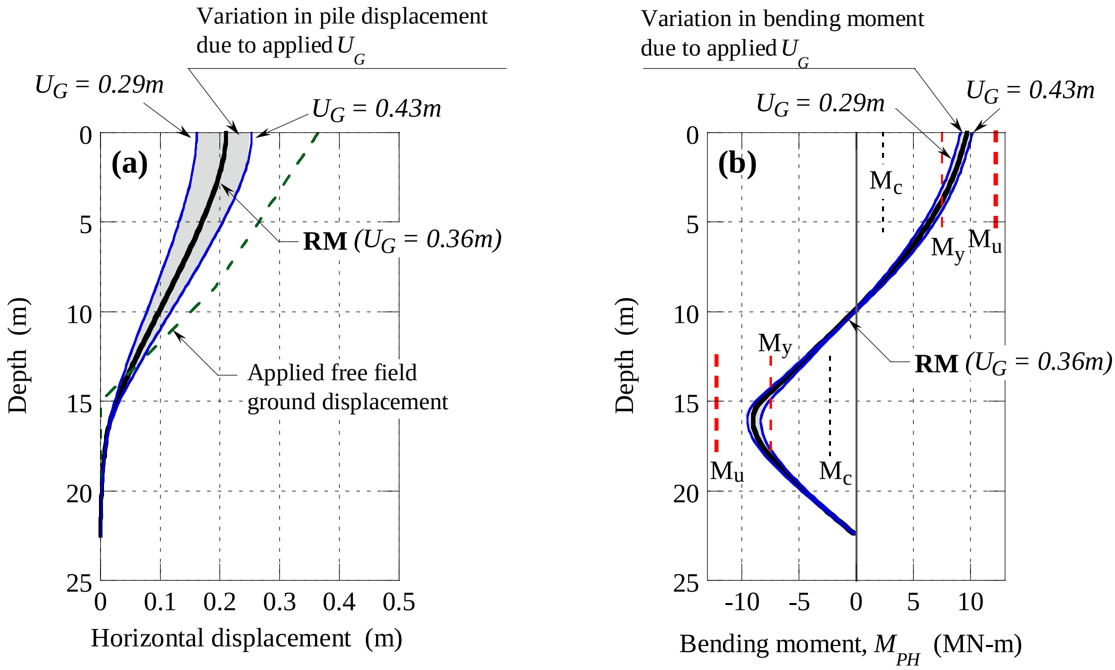

Figure 8. Effects of applied lateral ground displacement on the pile response computed in the pseudo-static analysis: (a) pile displacements; (b) bending moments

Discussion

The above results clearly illustrate a high sensitivity of the pile response on the parameters of the simplified model. This sensitivity is not specific to the adopted approach in this study, but rather is a common feature of simplified methods of analysis. It simply reflects the significant uncertainties associated with the complex phenomena considered and their gross simplification in the pseudo-static method of analysis. The results also clearly emphasize the need for a parametric evaluation of the seismic response when using simplified methods of analysis. In terms of the previously introduced assessment approaches, a deterministic approach including parametric analyses (DA_P) would be required when using simplified methods of analysis for seismic performance assessment.

In the current practice, various methods for simplified (pseudo-static) analysis are used. These methods are similar in principle however they all have distinct modelling features and use different load-deformation relationships, geotechnical data and empirical correlations. For this reason, they all require an independent process of 'calibration' in which model parameters will be rigorously examined and their relevant range of values identified. Note that this calibration is both model-specific and problem-specific. For example, the pseudo-static analysis method presented herein when applied to the assessment of piles subjected to lateral spreading will need different set of reference values for the model parameters, e.g. magnitude of U_G, load combination rule for U_G and F, and stiffness degradation factor β.

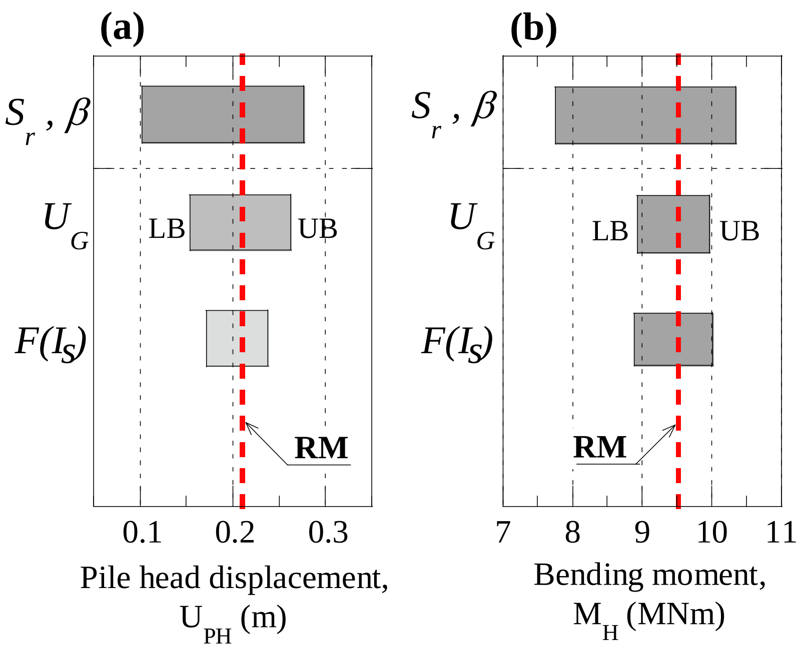

Figure 9. Tornado charts depicting pile response computed in parametric pseudo-static analyses: (a) pile-head displacement; (b) bending moment at pile head

6.4 SEISMIC EFFECTIVE STRESS ANALYSIS

Implementation steps

Unlike the simplified analysis procedure where the response of the pile is evaluated using a beam-spring 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. This analysis aims at a very detailed modelling of the ground response and soil-structure system in a rigorous dynamic analysis. The seismic effective stress analysis is difficult to implement in practice because it requires significant computational resources and specialists knowledge from the user. In concept, the effective stress analysis could be considered as the opposite approach to that of the practical pseudo-static analysis.

The implementation of the effective stress analysis generally involves three steps (Fig. 10):

(1) Determination of the parameters of the constitutive model
(2) Definition of the numerical model
(3) Dynamic analysis and interpretation of results.

In the first step, parameters of the constitutive model for the soil are determined using results from laboratory tests on soil samples and data from in-situ investigations. The required types of laboratory tests are model-specific and are generally used for determination of stress-strain relationships and effects of excess pore pressures on the soil response (liquefaction tests). Whereas most of the constitutive model parameters can be directly evaluated from data obtained from laboratory tests and in-situ investigations, some parameters are determined through a calibration process in which best-fit values for the parameters are identified in simulations of laboratory tests (so-called element test simulations).

In the second step, the numerical model is defined by selecting appropriate element types, dimensions of the model, mesh size, boundary conditions and initial stress state. The last two requirements often receive less attention, even though they have pivotal influence on the performance of the constitutive model and numerical analysis. Namely, one of the key advantages of the advanced numerical analysis is that no postulated failure and deformation modes are required, as these are predicted by the analysis itself. In this context, the selection of appropriate boundary conditions along end-boundaries and soil-foundation-structure interfaces are critically important in order to allow unconstrained response and development of relevant deformation modes. Similarly, stress-strain behaviour of soils and liquefaction resistance are strongly affected by the initial stress state of the soil, and therefore, an initial stress analysis is required to determine gravity-induced stresses in all elements of the model resembling those in the field.

In the final step, an acceleration time history (ground motion) is selected which is used as a base excitation for the model. Considering the geometry of the problem and anticipated behaviour, numerical parameters such as computational time increment, integration scheme and numerical damping are adopted, and the dynamic effective stress analysis is then executed. The analysis is quite demanding on the user in all steps including the final stages of post-processing and interpretation of results since it requires an in-depth understanding of the phenomena considered, constitutive model used and particular numerical procedures adopted in the analysis. 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 assessment of the seismic performance of important structures, high-quality geotechnical data from field investigations and laboratory tests are needed in order to model the particular deformational characteristics (stress-strain relationships) of the soils in questions. Such data are rarely available, however, and this has been often used as an excuse to avoid using the seismic effective stress analysis in geotechnical practice. However, even when conventional data is used as input, this analysis still provides an important and unique contribution in the seismic performance assessment of earth structures and soil-foundation-structure systems, as illustrated below.

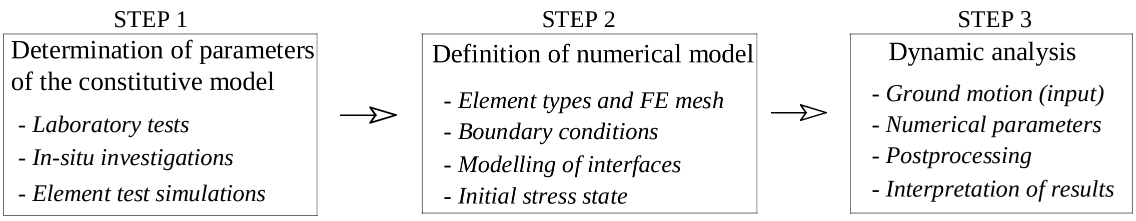

Figure 10. Key steps in the implementation of seismic effective stress analysis

Numerical model

The 2-D finite element model adopted for the effective stress analysis of the pile foundation of Fitzgerald Bridge is shown in Figure 11. The model includes the soil, pile foundation (both existing piles and new piles) and the superstructure. Four-node solid elements were employed for modelling the soil and bridge superstructure while beam elements were used for the piles and pile cap. Lateral boundaries of the model were tied to share identical displacements in order to simulate a free field ground motion near the boundaries. Along the soil-pile interface, the piles and the adjacent soil were connected at the nodes and were forced to share identical horizontal displacements.

The footing, bridge deck and pier were all modelled as linear elastic materials with an appropriate tributary mass to simulate inertial effects from the superstructure. Nonlinear behaviour of the piles 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, 1998a; 1998b). Details of the constitutive law and numerical procedures will not be discussed herein, but rather modelling of the liquefaction resistance based on conventional geotechnical data will be demonstrated.

The model shown in Figure 11 was subjected to an earthquake excitation with similar general attributes (magnitude, distance and PGA) to those relevant for the seismic hazard of Christchurch. An acceleration record obtained during the 1995 Kobe earthquake (M=7.2) was scaled to a peak acceleration of 0.4g and used as a base input motion. Needless to say, the adopted input motion is neither representative of the source mechanism nor path effects specific to Canterbury, but rather it was considered a relevant excitation typical for the size of the earthquake event considered in the analysis.

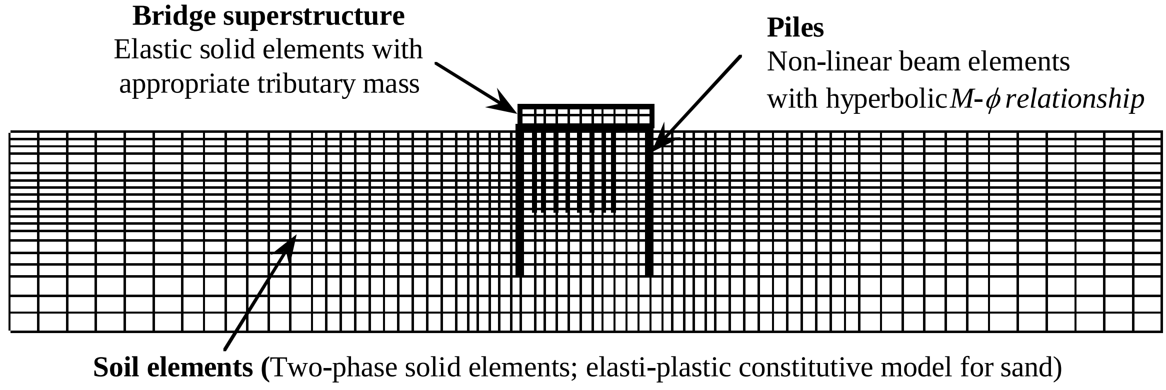

Figure 11. Numerical model used in the seismic effective stress analysis of Fitzgerald Bridge

Modelling of liquefaction resistance

For a rigorous determination of parameters of the employed constitutive model, about 15 to 20 laboratory tests are required including monotonic and cyclic, drained and undrained shear tests. In the absence of laboratory tests for the soils at the Fitzgerald Bridge site, the constitutive model parameters were determined by largely adopting the parameters of Toyoura sand (Cubrinovski and Ishihara, 1998a) and modifying the dilatancy parameters as described below.

Borelogs, penetration resistance data from CPTs and SPTs and conventional physical property tests were the only geotechnical data available for the soils at Fitzgerald Bridge site. 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 fines content and the magnitude of the earthquake (using magnitude scaling factor), these charts provided the cyclic stress ratios required to cause liquefaction in 15 cycles, which are shown by the solid symbols in Figure 12. Using these values as a target liquefaction resistance, the dilatancy parameters of the model were determined and the liquefaction resistance was simulated for the two layers, as indicated with the lines in Figure 12. These two lines represent the simulated liquefaction resistance curves for the soils with N_1 = 10 and N_1 = 15 respectively. To illustrate better this process, results of element test simulations for the sand with N_1 = 10 are shown in Figure 13 where effective stress paths and stress-strain curves are shown for three different cyclic stress ratios of 0.12, 0.18 and 0.30 respectively. The number of cycles required to cause liquefaction in these simulations and the corresponding stress ratios are indicated with open symbols in Figure 12, depicting the simulated liquefaction resistance. Thus, only conventional data were used for determination of model parameters. While this choice of material parameters practically eliminates the possibility for 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.

Computed ground response

Figure 14a 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 stress cycles until a complete liquefaction of this layer was reached 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 14b further illustrates the development of the excess pore water pressure throughout the depth of the deposit with time. Note that part of the steady

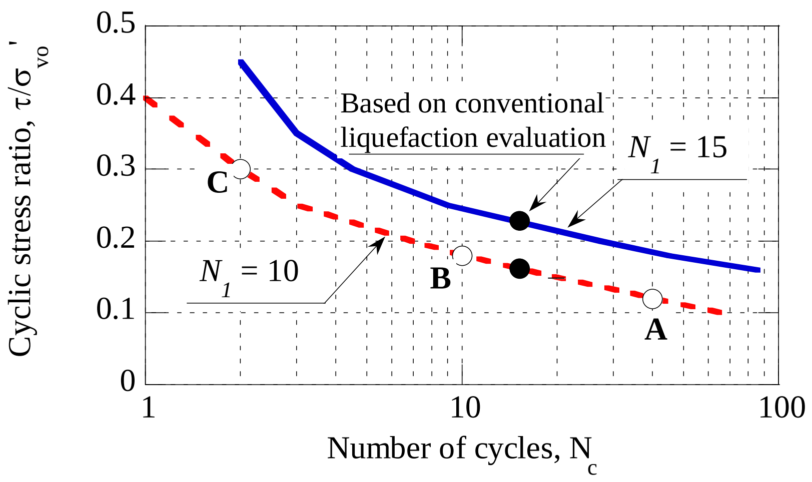

Figure 12. Liquefaction resistance curves adopted in the seismic effective stress analysis (curves represent model simulations)

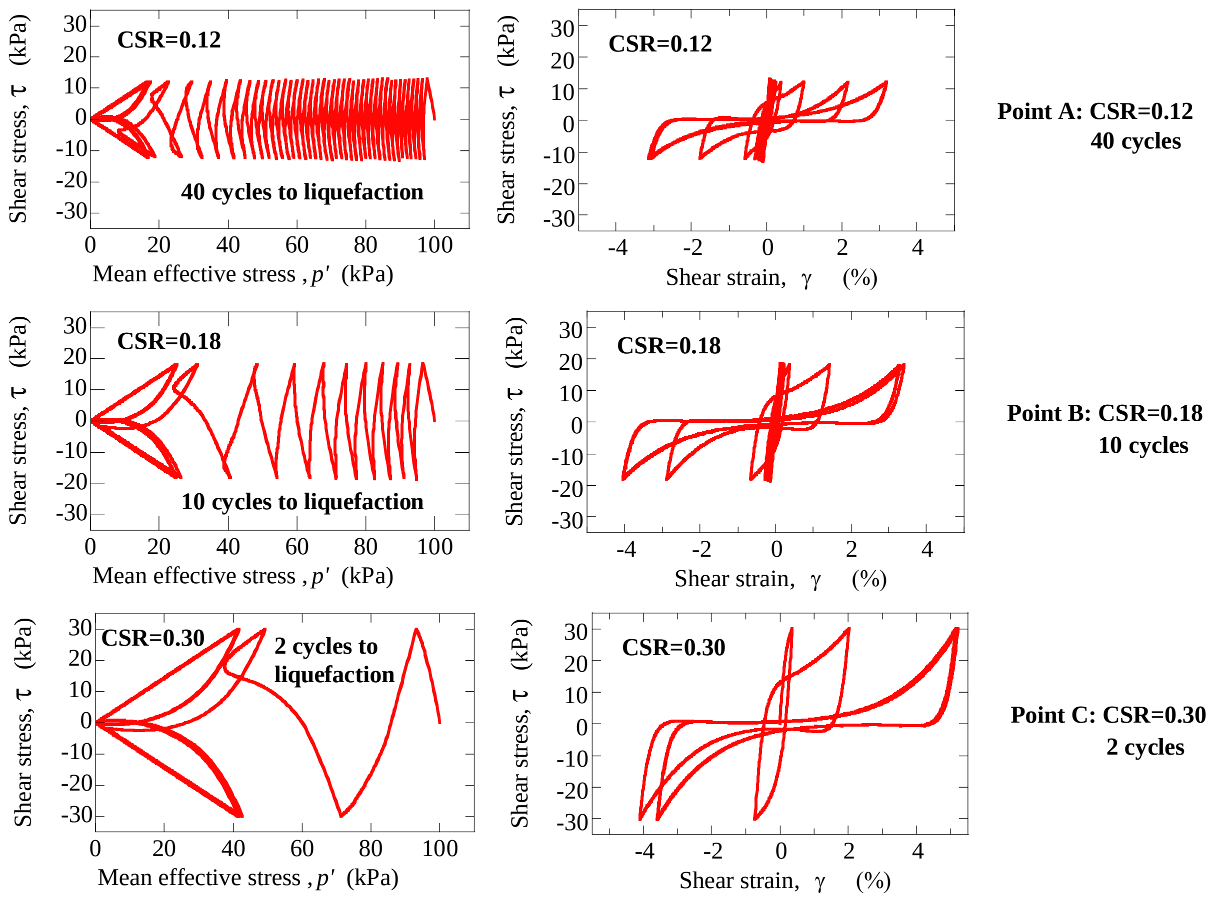

Figure 13. Effective stress paths and stress-strain curves obtained in element test simulations for the soil layer with N_1 = 10

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 outlined in Figure 14 will be reflected in the development of transient deformation and permanent displacements of the ground. The seismic effective stress analysis can simulate these complex features of the ground response and their effects on structures.

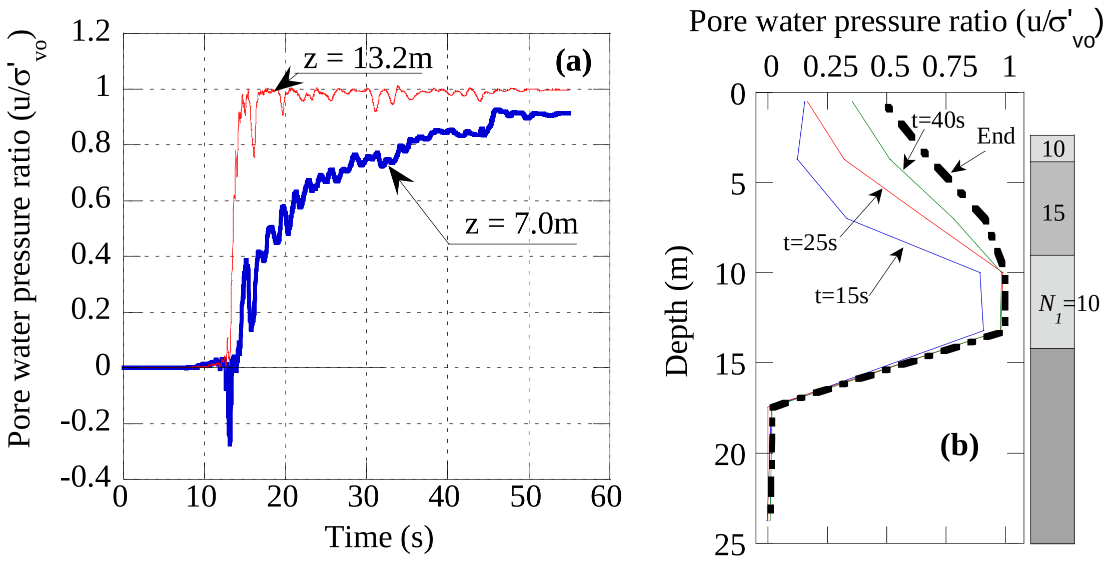

Figure 14. Computed excess pore water pressure in the free field soil: (a) time histories at mid-depths of layers with N_1 = 10 and N_1 = 15; (b) distribution of excess pore water pressures throughout the depth of the deposit and time

Computed pile response

The computed time history of horizontal displacement of the pile is shown in Figure 15a together with the corresponding displacement of the ground in the free field. The peak pile displacement reached about 0.18m at the pile head, which is significantly smaller than the peak free field displacement at the ground surface of 0.30m indicating relatively stiff pile behaviour (the pile is resisting the ground movement). The response shown in Figure 15a 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 15b). 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 is available for other components of the numerical model including the foundation soil, old and new piles, and response of the superstructure.

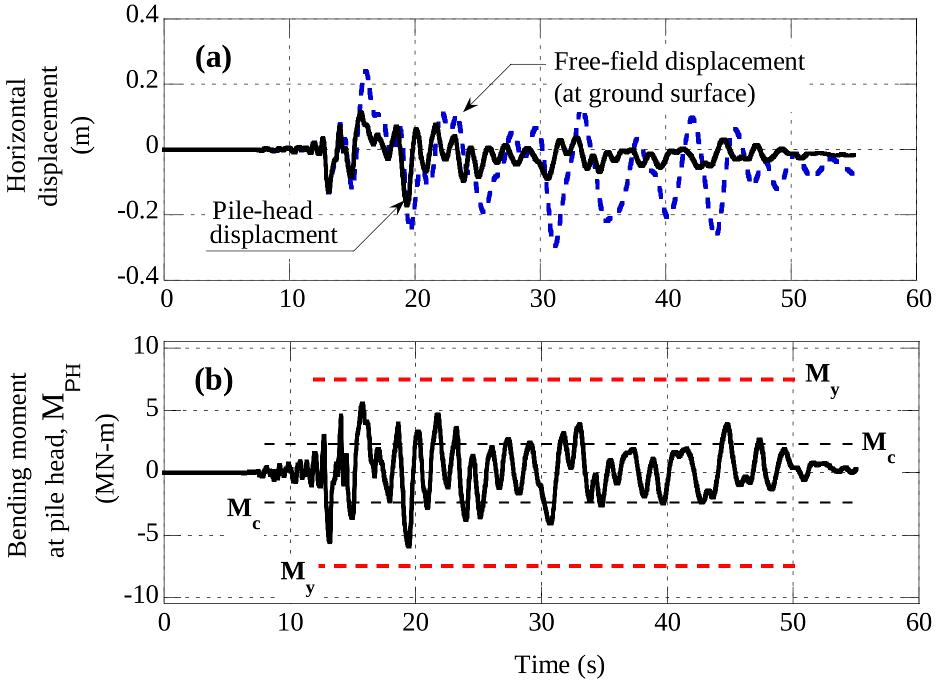

Figure 15. Computed response of the pile in seismic effective stress analysis: (a) horizontal displacement at pile head; (b) bending moment at pile head

Discussion

As illustrated in the above application, the seismic effective stress analysis allows realistic and detailedsimulation of the seismic response of geotechnical structures induced by strong earthquakes. 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. The analysis permits a rigorous assessment of the seismic performance of the soil-structure system as a whole and each of its components.

Effects of excess pore water pressure are often a key factor in the seismic response of ground and earth structures. Hence, the ability of this analysis to capture details of pore pressure build-up, development of liquefaction and consequent loss of strength and stiffness in the soil is of great value. The method simulates the most salient features of seismic behaviour of soils including peculiar effects from individual layers and cross interaction amongst them such as "base-isolation effects" or progressive liquefaction due to upward flow of water.

Because of its complexity and high-demands on the user, the seismic effective stress analysis is typically applied in a deterministic fashion using a single scenario (DA) or input ground motion. However, this analysis also provides an excellent tool for assessment of alternative design solutions, effectiveness of structural strengthening and soil remediation (countermeasures against liquefaction) on a comparative basis by quantifying their effects on the ground deformation, structural response and reduction (control) of damage.

6.2 PROBABILISTIC APPROACH

Background

A probabilistic approach (PEER framework) for Performance-Based Earthquake Engineering (PBEE) has been recently developed for a robust assessment of seismic performance of structures (Cornell and Krawinkler, 2000; Krawinkler 1999). This approach employs an integrated probabilistic treatment of all uncertainties that apply to the prediction of ground motion and evaluation of system response and associated damage (uncertainties associated with characteristics of ground motion, material properties, modelling approximations, seismic response and associated physical damage for a given response measure). Hence, it provides an alternative and more rigorous way for assessment of seismic performance of engineering 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 implementation of this procedure will be outlined in the following using the case study considered.

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 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 paths of seismic waves and surface-soil effects. In order to account for these uncertainties in the ground motion characteristics, the following procedure was adopted.

A suite of 40 ground motions recorded during strong earthquakes was first selected, as indicated in Figure 16a. 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 generated in this way, as indicated in Figure 16b, having very different amplitudes, frequency content and duration. Using each of these time histories as a base input motion, 400 effective stress analyses were conducted using the model shown in Figure 11 and procedures outlined earlier, as schematically depicted in Figure 16c.

Computed response

The next challenge to overcome is how to present results from 400 time history analyses in a meaningful way. Obviously, some relaxation in the rigorous treatment of time histories and evaluation of the response is needed here. In the probabilistic PBEE approach, this is achieved through the following reasoning:

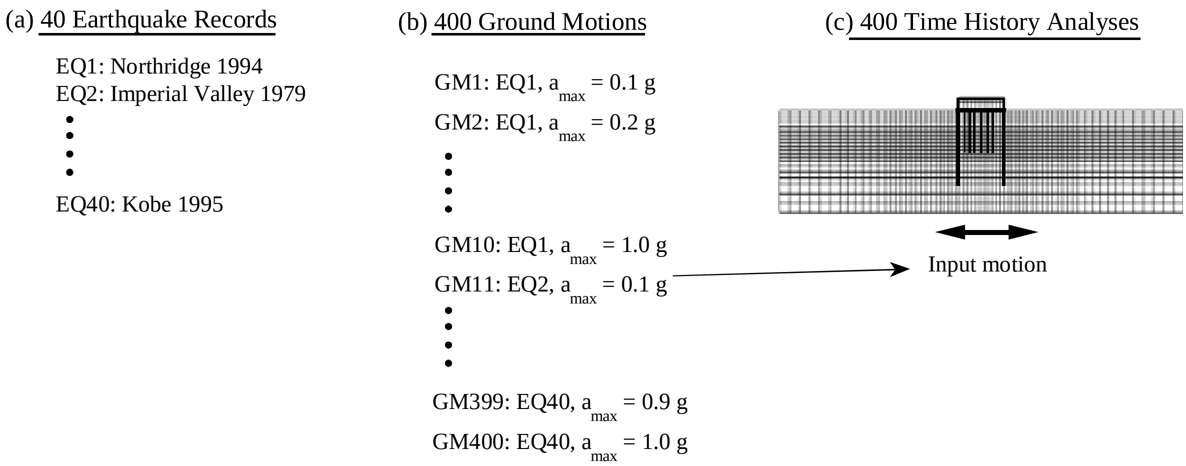

Figure 16. Schematic illustration of multiple effective stress analyses used in the probabilistic approach

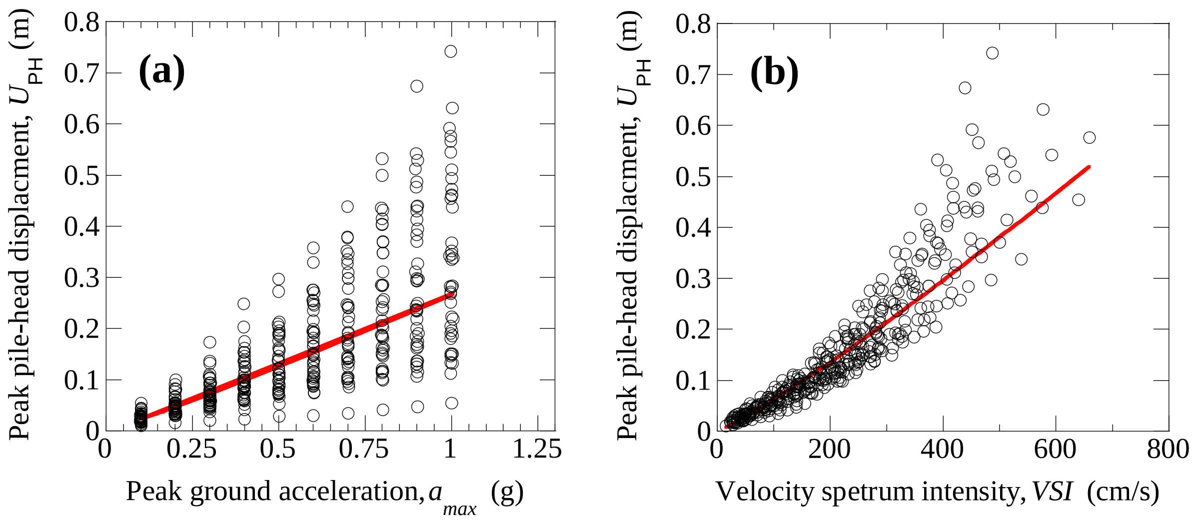

Figure 17. Computed pile-head displacements (U_{PH}) in 400 effective stress analyses: (a) correlation between (U_{PH}) and a_{max} of input motion; (b) correlation between (U_{PH}) and velocity spectrum intensity (VSI) of input motion

(1) First, the object of assessment is identified. Thus, instead of examining the entire soil-pile-structure system, for example, the attention is focused on the response of the pile.

(2) Next, a representative measure for the response of the pile is identified, i.e. a parameter that describes and quantifies the pile response efficiently ("Engineering Demand Parameter", EDP in the PBEE terminology). Hence, instead of using the entire time history of the pile response, the peak value of the response parameter (EDP) is used as a measure for the size of the response.

(3) Similarly, a single parameter is used to describe the input motion or measure the intensity of the ground motion ("Intensity Measure", IM).

(4) Finally, the results of the analyses are presented by correlating the parameter representing the size of the response (EDP) with the intensity of the ground motion (IM).

For example, one way of presenting the results from the 400 analyses with respect to the pile response for Fitzgerald Bridge is shown in Figure 17a where the peak displacement at 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 (*EDP*) while a_{max} is a measure for the intensity of the ground motion (IM). Each open symbol in Figure 17a represents the result (peak response of the pile) from one of the 400 seismic effective stress analyses while the solid line is an approximation of the trend from a regression analysis.

The scatter of the data in Figure 17a 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 (input PGA). 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 intensity measure that is appropriate for all problems but rather the intensity measure is problem-dependent and is affected by the particular deformational mechanism and features of the phenomena considered. Based on detailed numerical studies, Bradley et al. (2008) have identified that velocity-based intensity measures correlate the best with the seismic response of piles, and that in particular the velocity spectrum intensity (*VSI*) is the most efficient intensity measure for piles. This is illustrated in Figure 17b where the same results for U_{PH} from the 400 analyses shown in Figure 17a are re-plotted using *VSI* as the 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 17 provide means for estimating the peak response of the piles of Fitzgerald Bridge for all levels of earthquake excitation, from elastic response to failure.

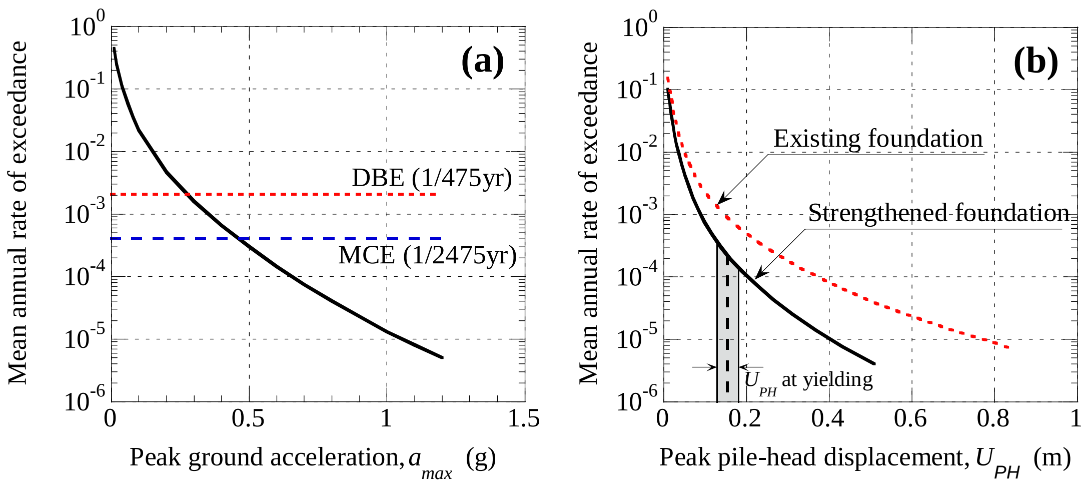

Figure 18. Probabilistic assessment of seismic performance of pile foundation: (a) seismic hazard curve for Christchurch; (b) Demand hazard curve for piles of Fitzgerald Bridge

Assessment of seismic performance: Demand hazard curve

A conventional output from 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 18a 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).

By combining the seismic hazard curve expressed in terms of a_{max} (Fig. 18a) and the correlation between the peak pile response (U_{PH}) and a_{max} established from the results of the effective stress analyses (Fig. 17a), a so-called "Demand Hazard Curve" was produced, shown in Figure 18b for the existing and new piles respectively. In this way, the probability for exceedance of a certain level of peak pile displacement in any given year (annual rate of exceedance) could be estimated for the piles of Fitzgerald Bridge. 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 and associated uncertainties in the characterization of the ground motion.

In the above interpretation, the peak pile displacement was adopted as a measure for the size of the pile response because it is a good indicator of the peak deformation and 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 (the peak curvature of the pile), and then the demand hazard curve can be easily expressed in terms of a damage measure, thus providing likelihood of characteristic damage levels for the pile (cracking, yielding, failure). Furthermore, the physical damage of the pile foundation will lead to losses, and hence, 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, and will facilitate communication of the design outside the profession. Clearly, the probabilistic assessment provides alternative measures of the seismic performance of the pile while rigorously accounting for the uncertainties associated with the seismic hazard and phenomena considered. This approach can be applied to seismic performance assessment of any other component of the soil-pile-structure system and to the bridge as a whole. Also, other sources of uncertainty such as those related to modelling, soil and site characterization can be easily incorporated in the analysis and their effects on the response can be quantified.

6.5 SUMMARY AND CONCLUSIONS

Three different approaches for assessment of the seismic performance of earth structures and soil-structure systems 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 associated damage; however, they focus on different aspects and make different contribution in the assessment. Key features of the examined approaches and their specific contribution in the seismic performance assessment are summarized in Table 3.

Pseudo-static analysis

The pseudo-static analysis is a practical approach based on conventional geotechnical data, engineering concepts and relatively simple computational models. It postulates a specific deformational mechanism and aims at estimating the peak response of the pile due to an earthquake under the assumption that dynamic loads can be represented as static actions. The method is easy to implement in practice and provides a suitable tool for evaluation of the seismic response of piles and associated damage to piles. This approach focuses on the pile itself (enhances foundation design) while it ignores the response of the system and other components of the system.

In addition to the uncertainties associated with the complex seismic behaviour and ground motion, there are significant uncertainties related to modelling arising from unknown variables and inaccurate model form. These modelling uncertainties are very pronounced in the simplified analysis because of the significant approximations and gross simplification of the problem adopted in this approach. Thus, when using simplified methods of analysis in the assessment, it is critically important to address these uncertainties through systematic parametric studies.

Seismic effective stress analysis

The seismic effective stress analysis aims at a very realistic simulation of the seismic behaviour of earth structures and soil-structure systems. It incorporates sophisticated nonlinear models for the soil, foundation and structure in a rigorous dynamic analysis. The key contribution of this analysis is that it allows examining in detail the performance of the soil-structure system under a strong earthquake excitation. Even results from a single analysis (such as that presented herein) illustrate the benefit of a detailed soil-pile-structure analysis.

The experience from recent strong earthquakes suggests that design concepts in which pile foundations are considered to remain within the elastic range of deformation during strong earthquakes are not economical. The PBEE philosophy also suggests accepting damage in seismic events, if this proves the most economic solution (Krawinkler, 1999). Hence, there is a need to consider inelastic deformation concurrently in both the superstructure and pile foundation, and to assess the performance both on a system level and at a component level (Gazetas and Mylonakis, 1998). Advanced numerical analyses provide this capability and methods based on the effective stress principle further permit consideration of important ground response features such as effects of excess pore pressures and liquefaction.

Since this approach focuses on a detailed evaluation of the seismic response, it is not appropriate for parametric evaluation including large number of analyses. In this context, the selection of an appropriate input motion is problematic in cases when rigorous assessment and quantification of the seismic performance of important structures is needed.

Probabilistic approach

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. It allows us to combine geotechnical and structural design aspects and to evaluate their effects on the performance of the entire system (soil-foundation-structure system) and each of its components. It is worth noting that in spite of the use of an effective stress analysis as a basic computational tool in the probabilistic approach employed herein, details of the response were not considered in the seismic performance assessment.

Future needs

The examined approaches address different aspects in the assessment and, in essence, are complimentary in nature. It is envisioned that these approaches will be used in parallel in the future, and hence, they all require further development and improvement. The pseudo-static approach requires establishment of improved models depicting multiple deformational mechanisms and in particular more rigorous and systematic procedures for parametric evaluation of the seismic response. Methods based on seismic effective stress analysis require improvement in the simulation of large ground deformation and more emphasis on use of sophisticated nonlinear models for an integrated analysis of the soil-foundation-structure system. Finally, further development of the probabilistic approach is needed including efforts towards simplification of procedures and identification of representative response measures (EDPs) and ground motion measures (IMs) for various specific problems.

All of these analysis procedures improve our understanding of complex seismic behaviour and enhance engineering judgement, which is probably one of the most significant contributions that one can expect from such an exercise.

Acknowledgements

The authors would like to acknowledge the financial support provided by the Earthquake Commission (EQC), New Zealand.

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Appendix to Chapter 5: Crust-Inertial Force Plots

Each of the twenty four plots included in this appendix corresponds to a single pile (S, M, or F), a single cyclic ground displacement (0.1, 0.2, 0.3, or 0.4 m), and a particular density of liquefied (and crust) soil (either loose or medium-dense). The charts are presented four to a page, grouped by soil-pile combination, allowing the response of the pile to different displacement demands to be easily compared.

The figure below can be used to infer the mechanism of soil-pile interaction for each of the data point, based on its colour.

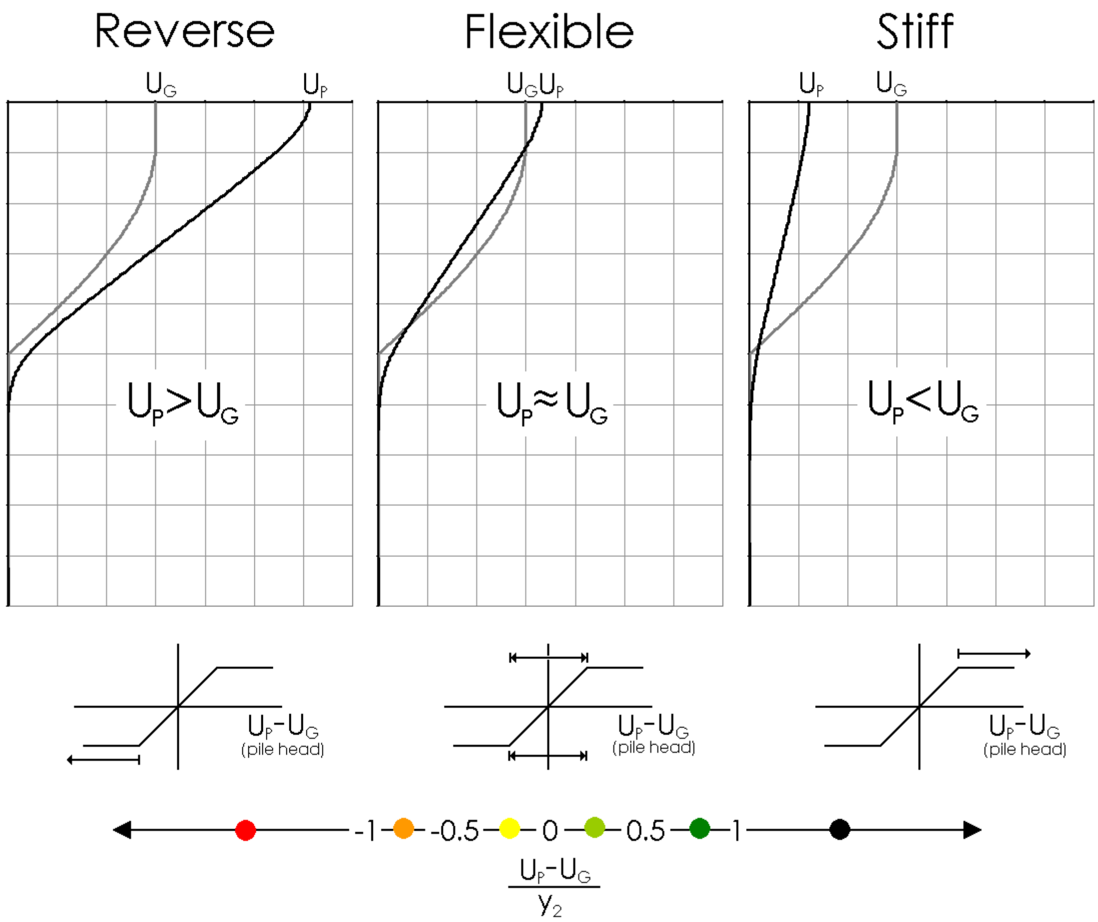

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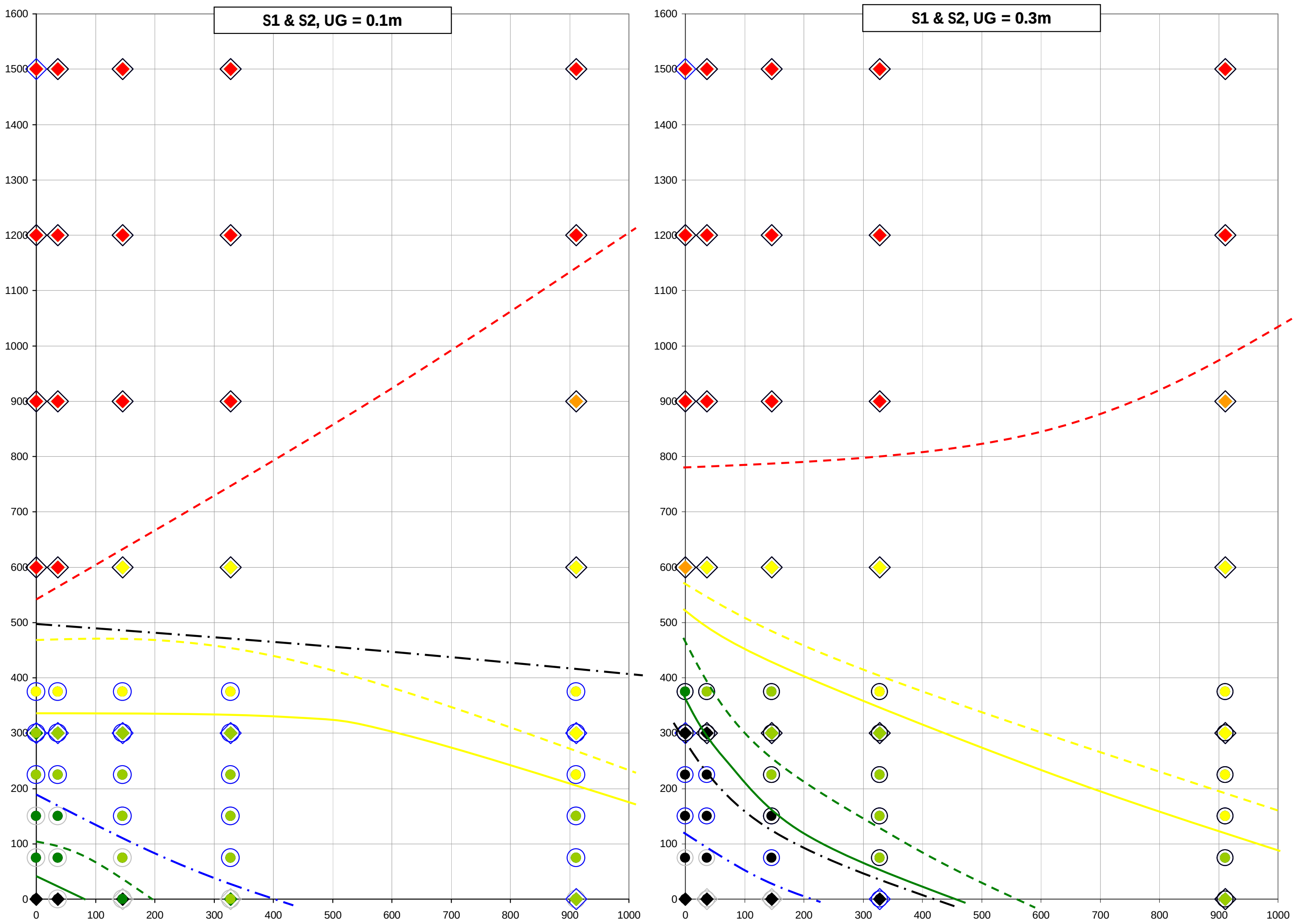
Figure Definition of stiff, flexible, and reverse mechanisms of soil-pile interaction used for the cyclic inertial force study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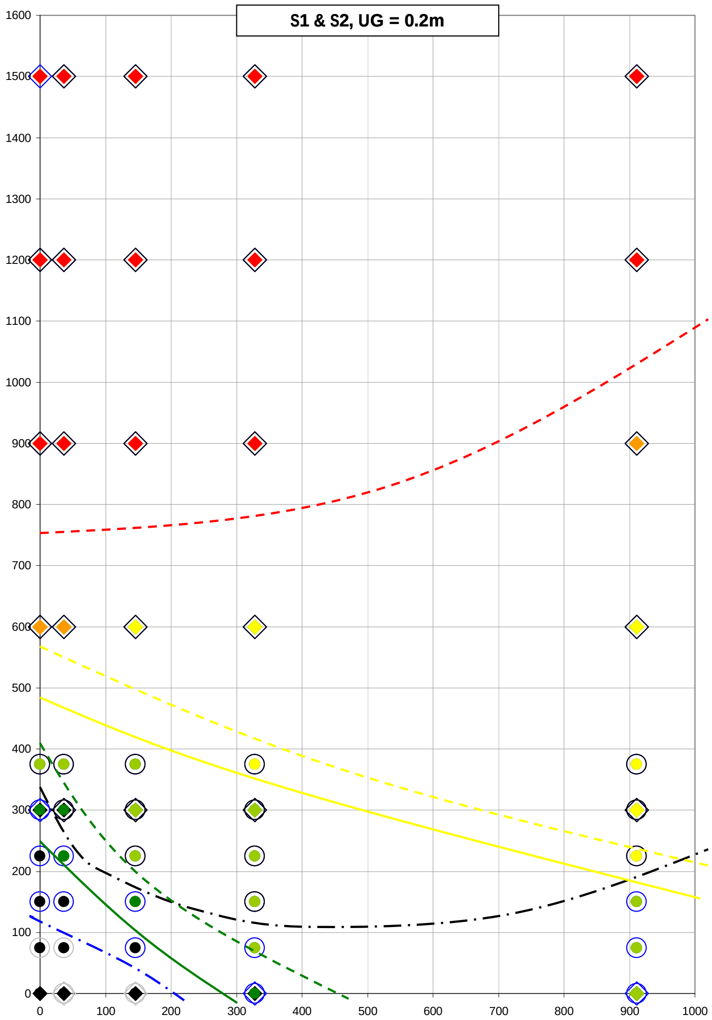
S1 & S2, UG = 0.3m


S1 & S2, UG = 0.2m
1600
1500
1400
1300
1200
1100
1000
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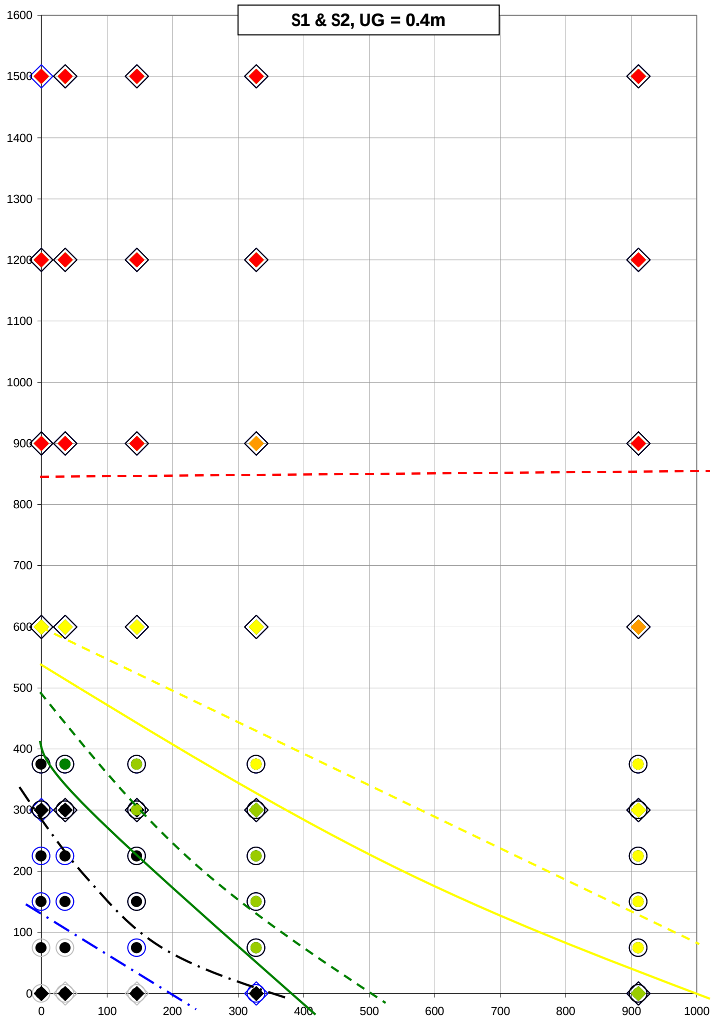
S1 & S2, UG = 0.4m
1600
1500
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1300
1200
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900
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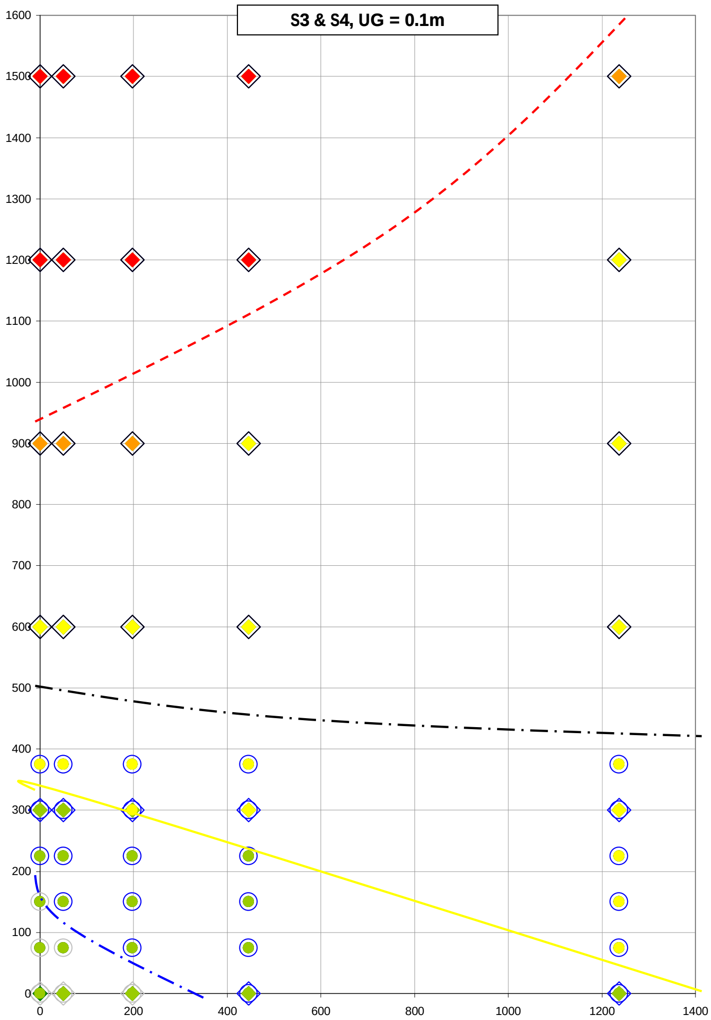
S3 & S4, UG = 0.1m
1600
1500
1400
1300
1200
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200
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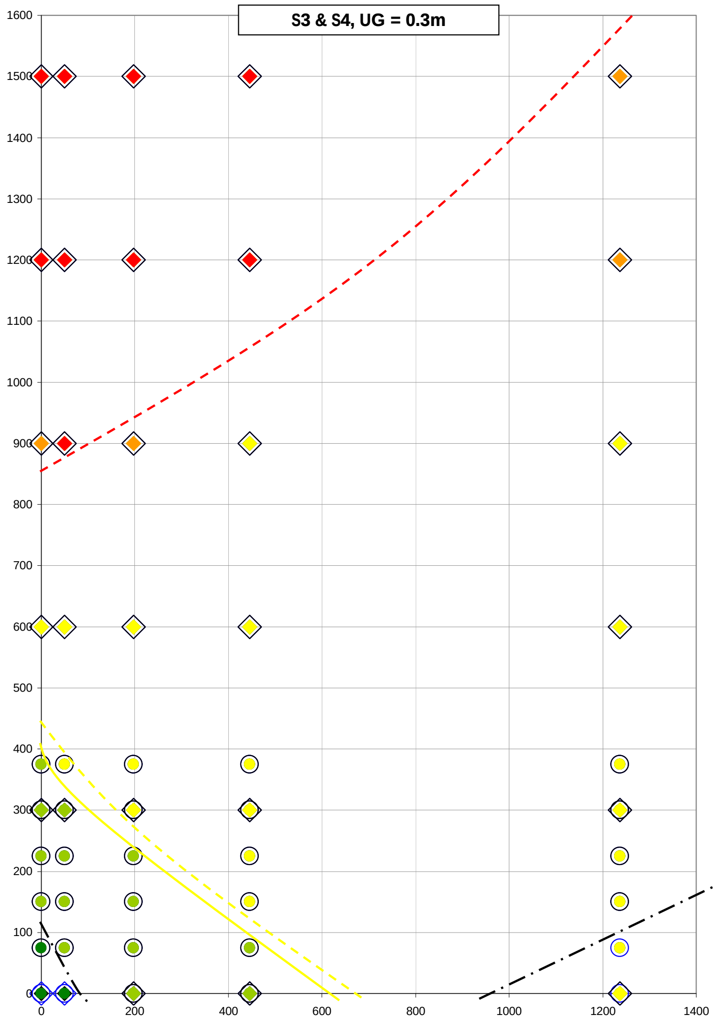
1000
1200
1400


S3 & S4, UG = 0.3m
1600
1500
1400
1300
1200
1100
1000
900
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700
600
500
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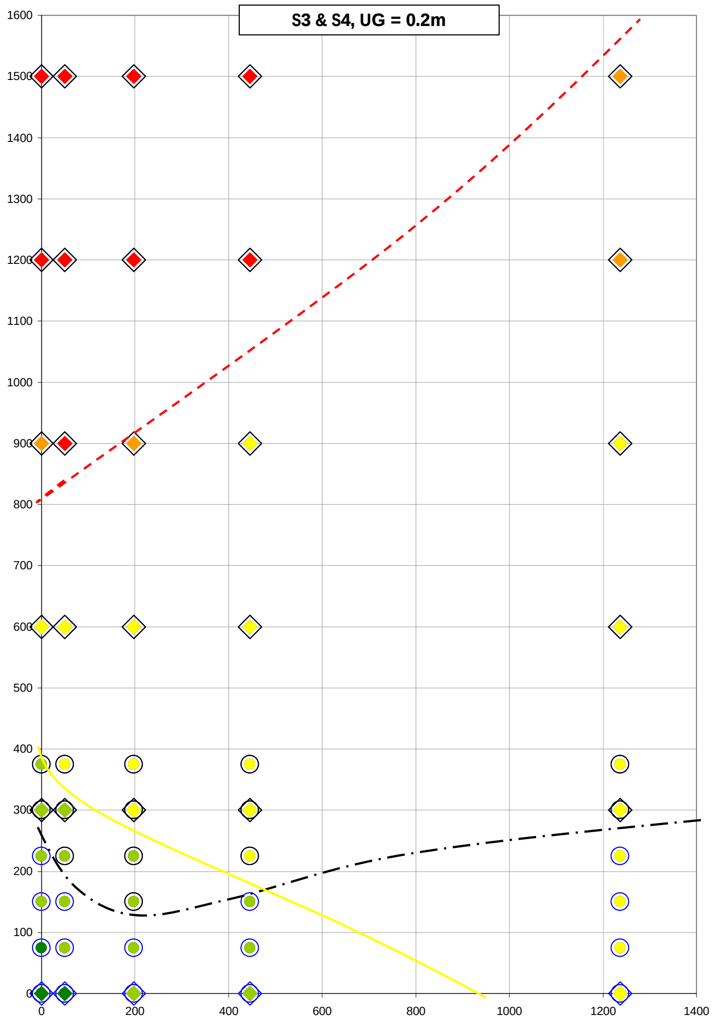
200
100
0
0
200
400
600
800
1000
1200
1400


S3 & S4, UG = 0.2m
1600
1500
1400
1300
1200
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200
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600
800
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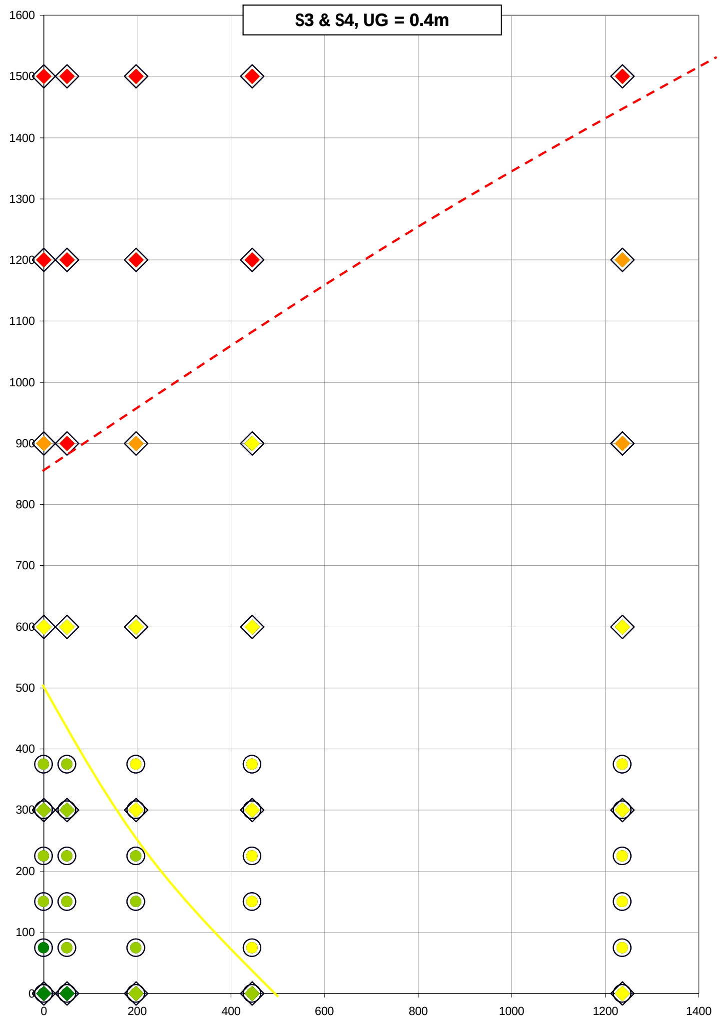
1200
1400


S3 & S4, UG = 0.4m
1600
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1400
1300
1200
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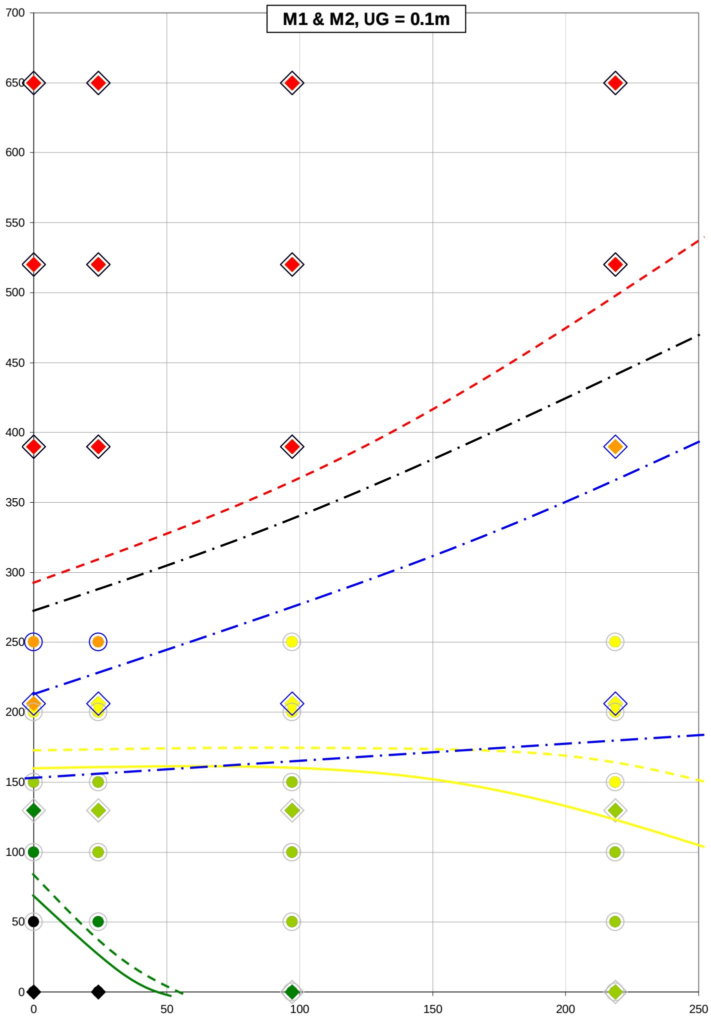
1200
1400


M1 & M2, UG = 0.1m
700
650
600
550
500
450
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250
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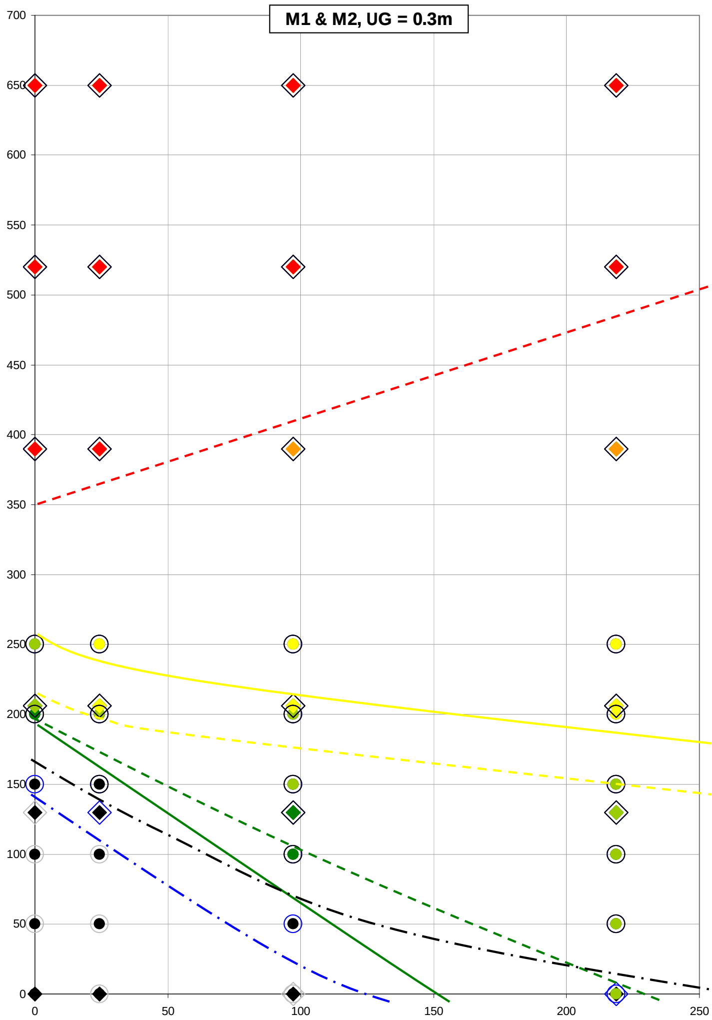
50
100
150
200
250


M1 & M2, UG = 0.3m
700
650
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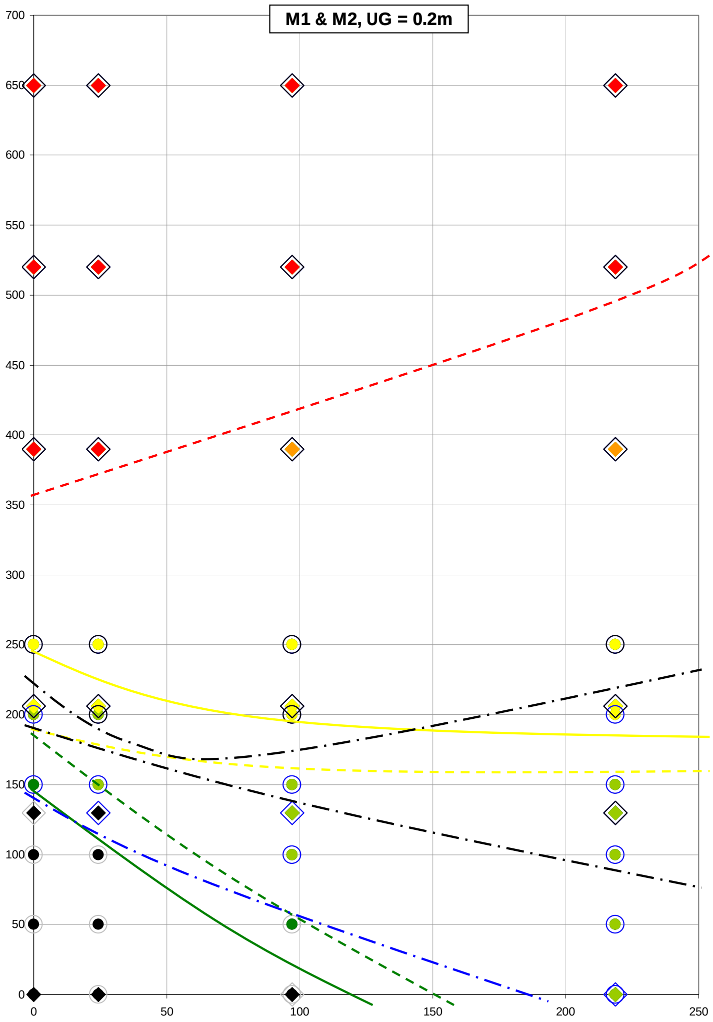
M1 & M2, UG = 0.2m
700
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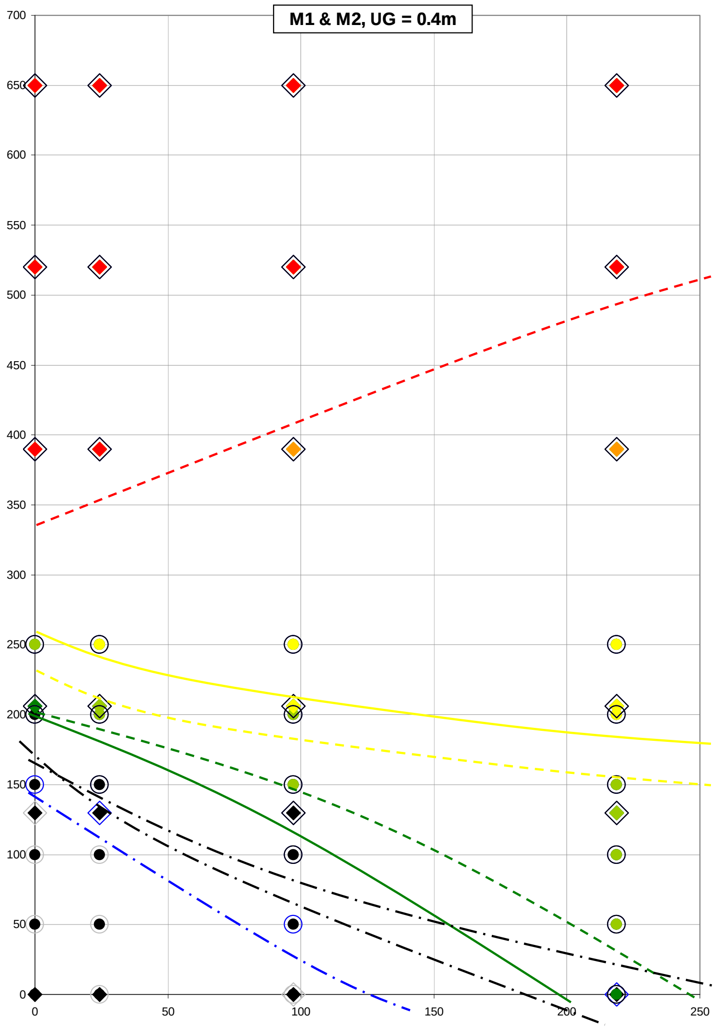
M1 & M2, UG = 0.4m
700
650
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550
500
450
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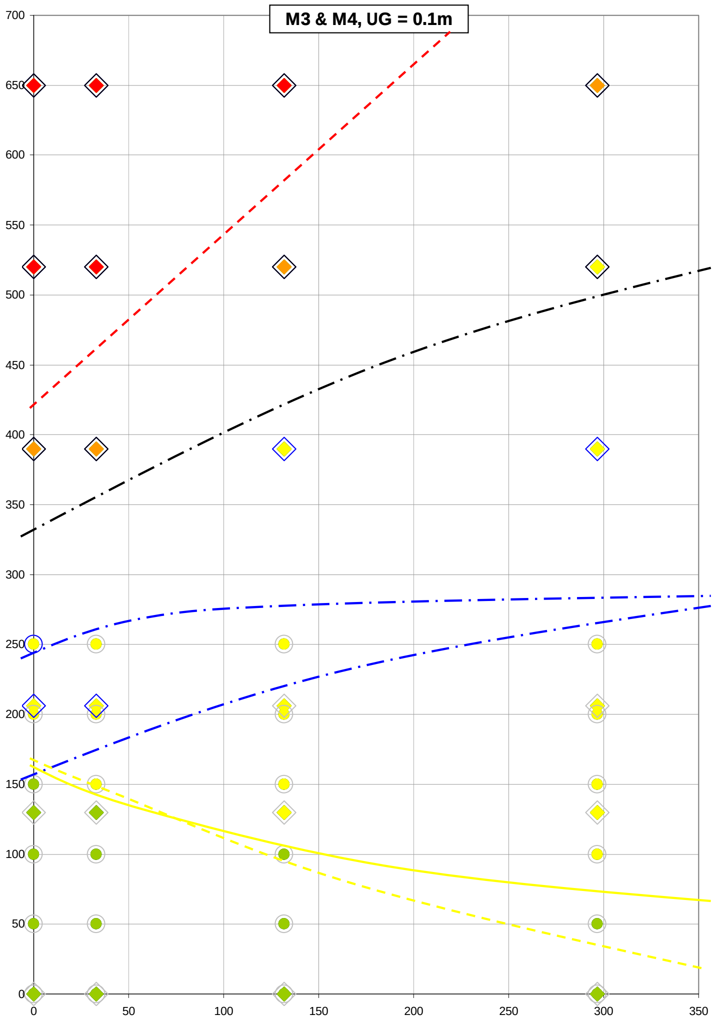
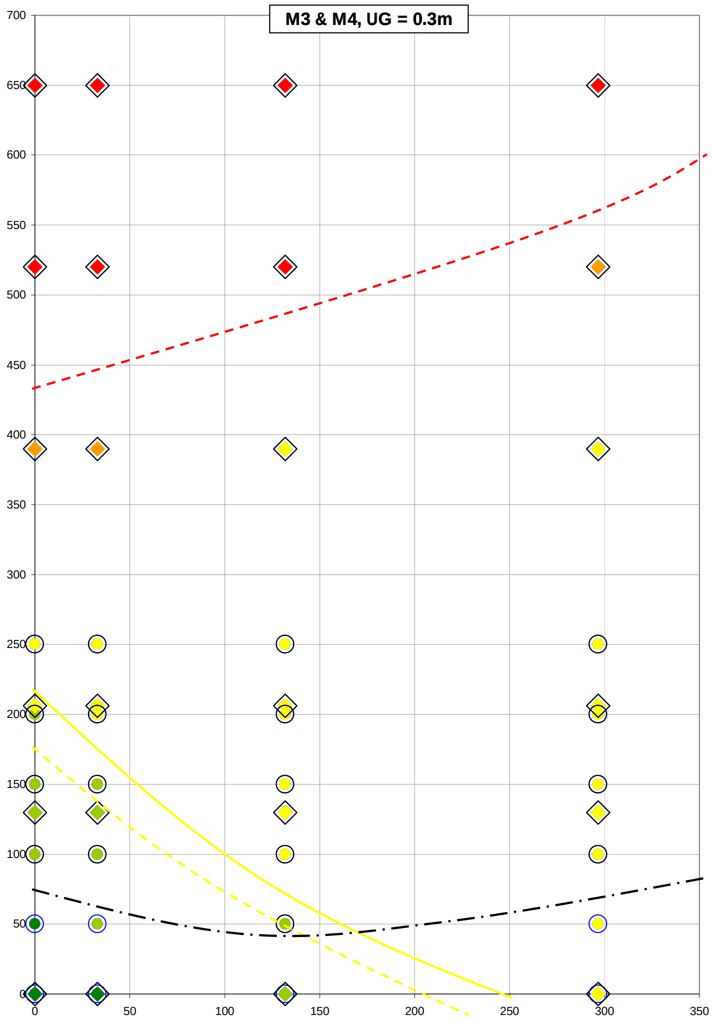
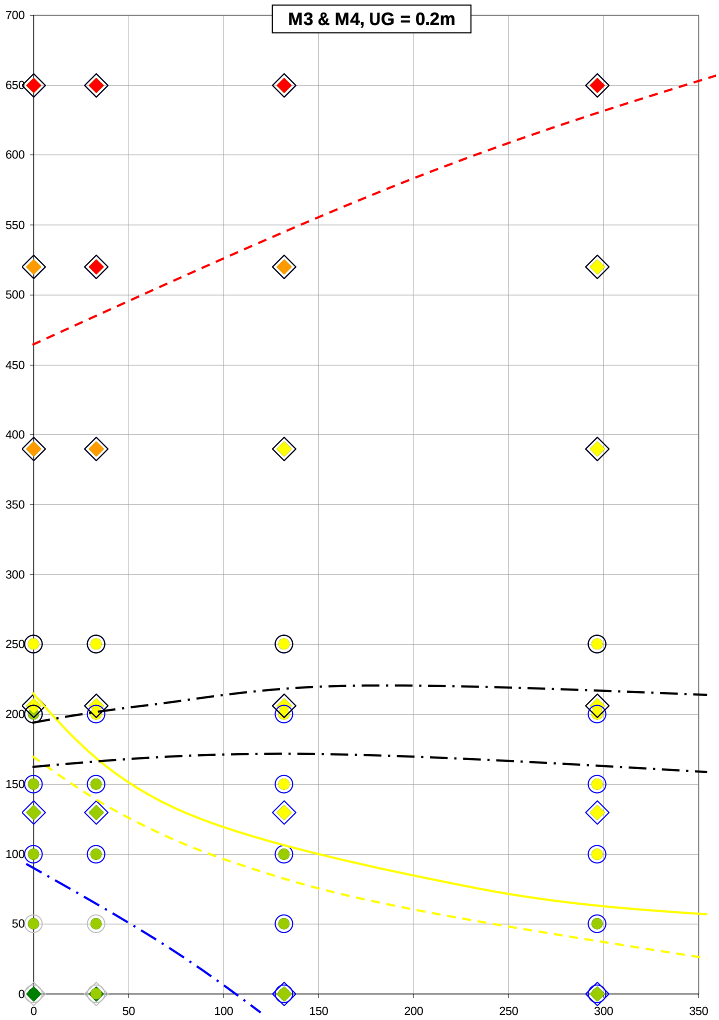

M3 & M4, UG = 0.1m


M3 & M4, UG = 0.3m


M3 & M4, UG = 0.2m
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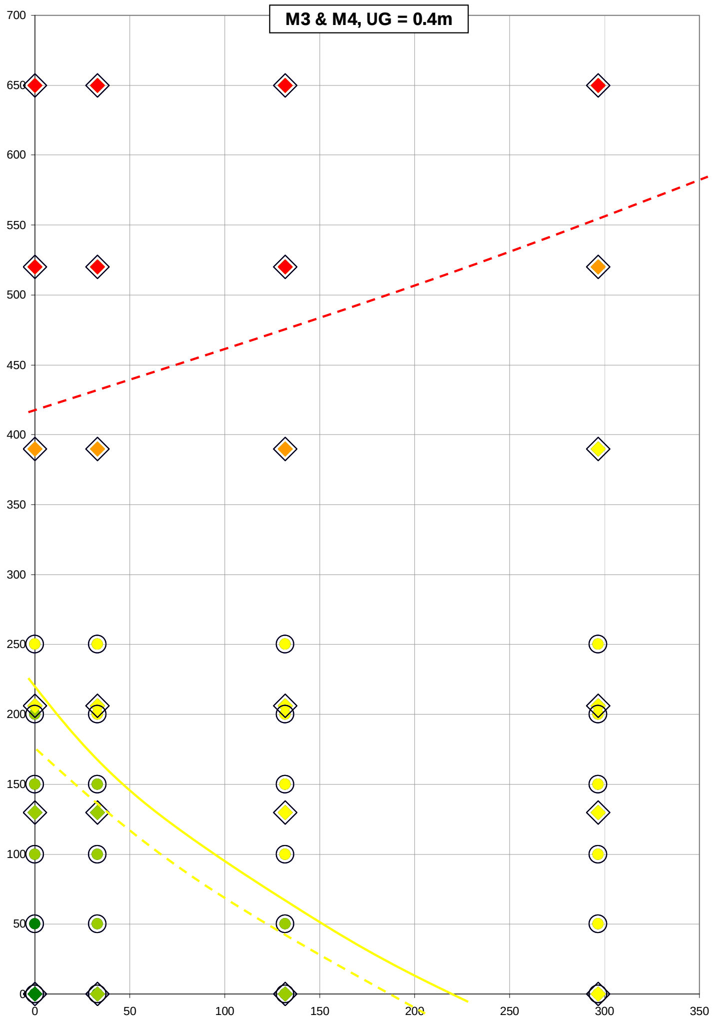
M3 & M4, UG = 0.4m
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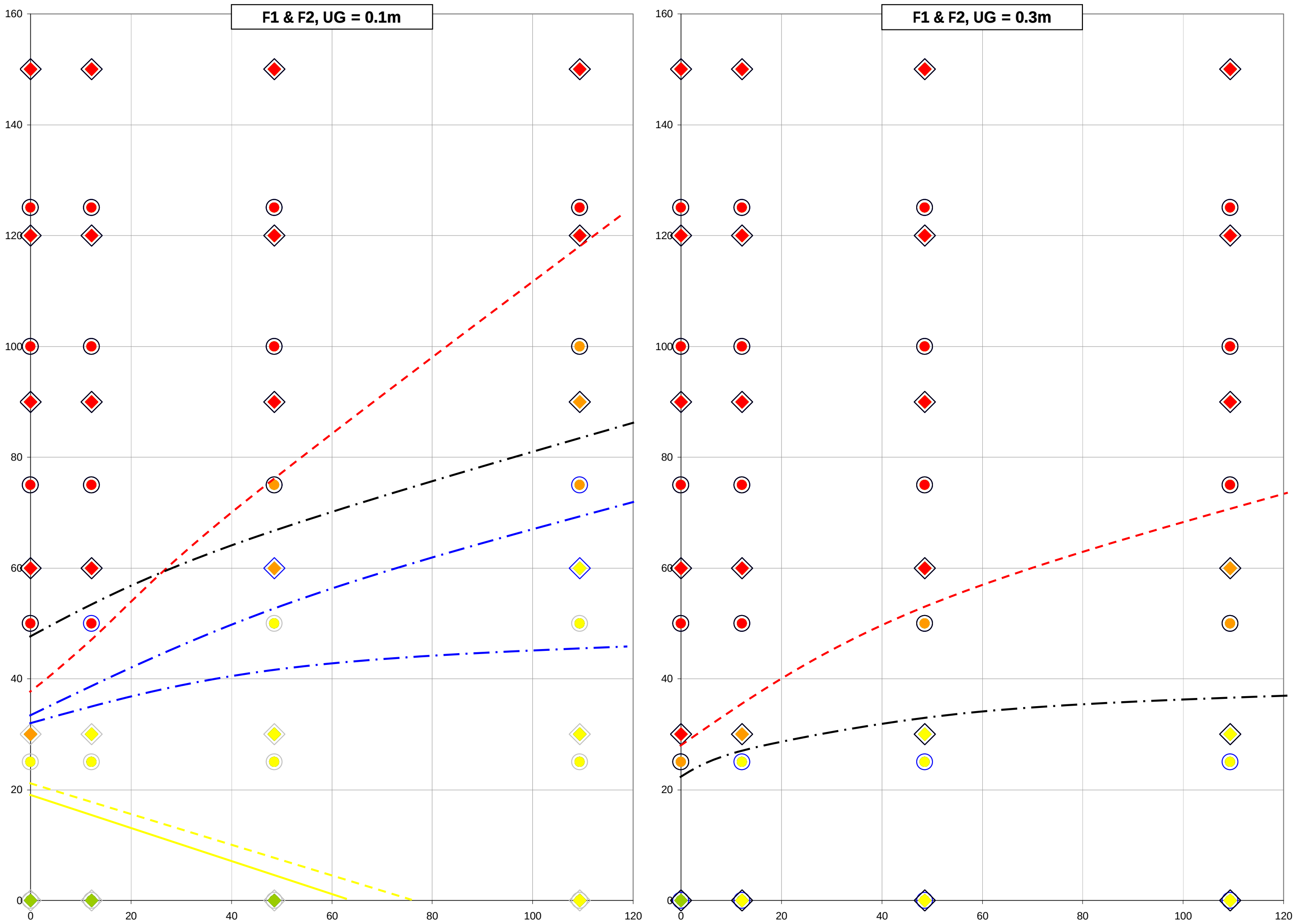
F1 & F2, UG = 0.1m
F1 & F2, UG = 0.3m
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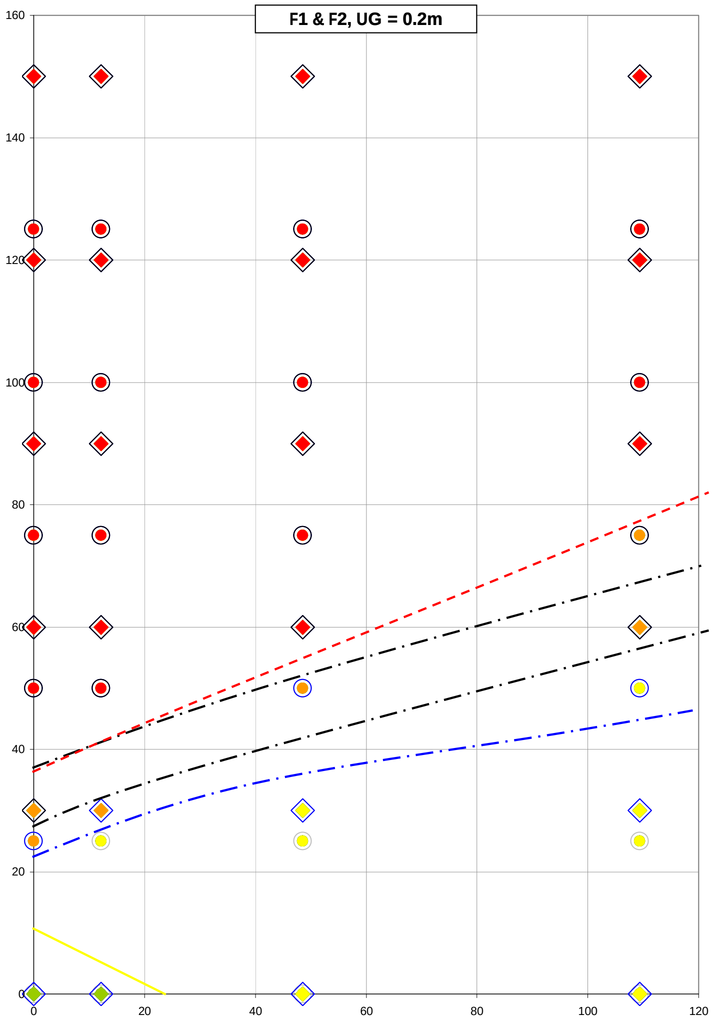
F1 & F2, UG = 0.2m
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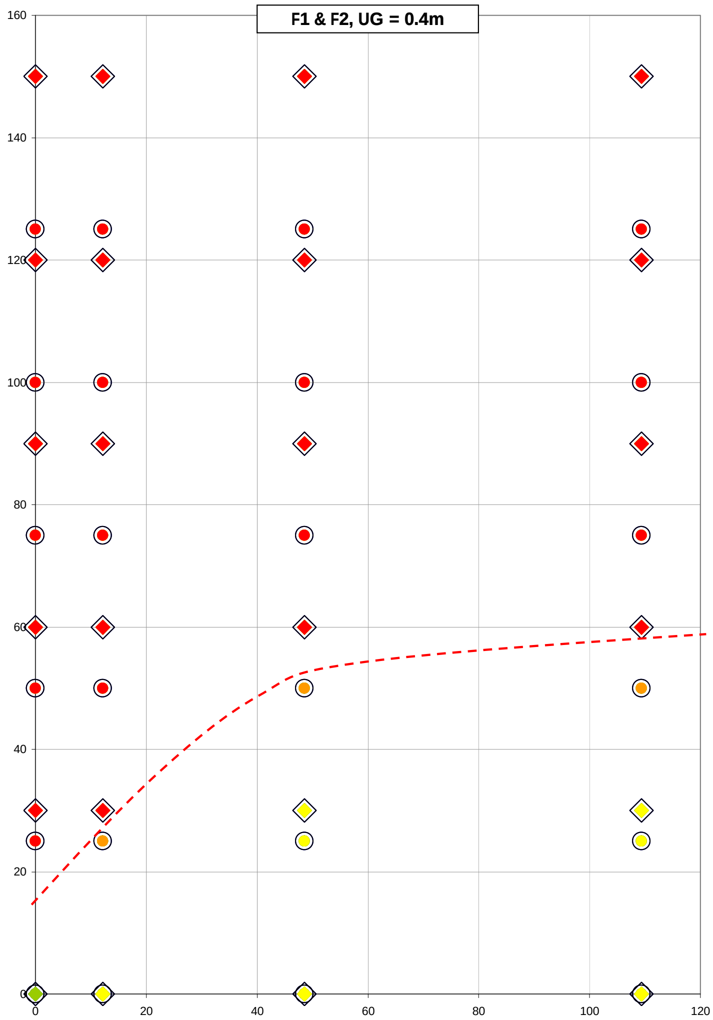
F1 & F2, UG = 0.4m
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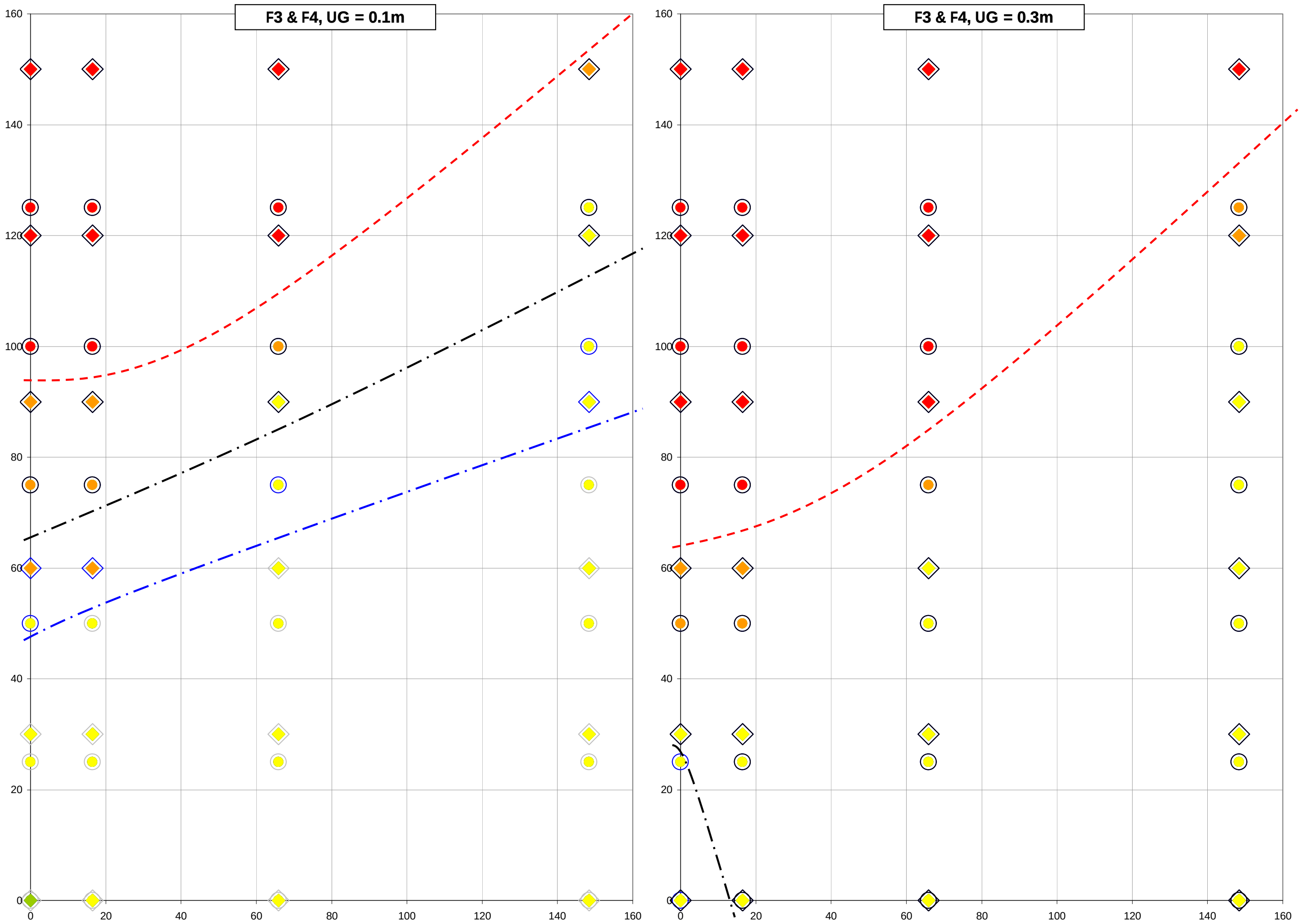
F3 & F4, UG = 0.1m
F3 & F4, UG = 0.3m
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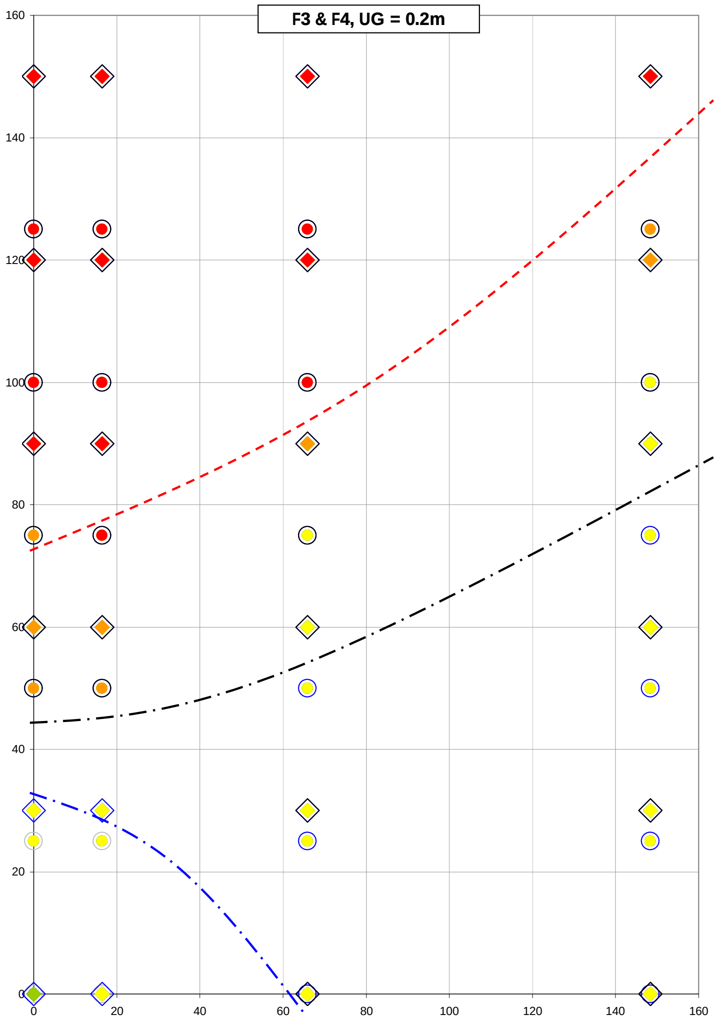
F3 & F4, UG = 0.2m
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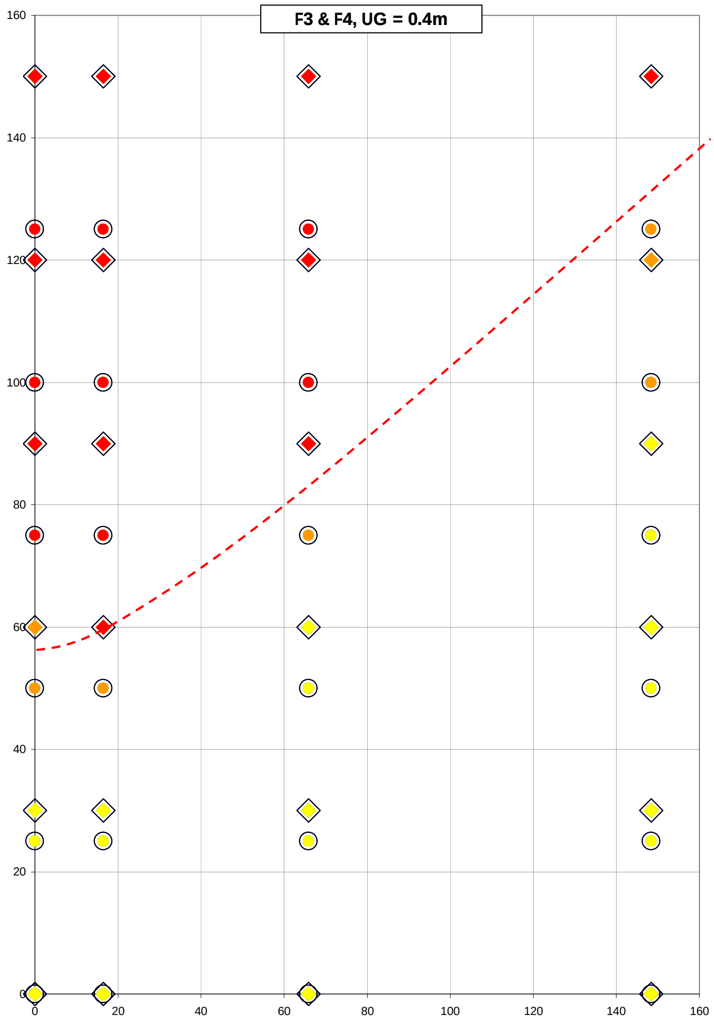
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