Integrated design of structure - foundation systems: the current situation and emerging challenges

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ABSTRACT: Nobody disputes the fact that a structure and the supporting foundation form a single entity. This paper reports on a research programme promoting the idea that the design process needs to reflect closer interaction between the structural and geotechnical teams; an idea which the author has been promoting for more than ten years and which recently received support from the Canterbury Earthquakes Royal Commission (recommendation 53).

The paper reviews work done to date on shallow and deep foundations, the results of experimental work, and insights gained from numerical analysis. Soil-structure-interaction (SSI) is frequently appealed to as factor in the earthquake response of structure-foundation systems, however, when this is based on elastic behaviour of the soil any effect is often minimal. To obtain real benefit some nonlinear soil behaviour must be mobilised; this is referred to herein as soil-foundation-structure-interaction (SFSI).

This paper presents an approach to including nonlinear soil-foundation-structure interaction (SFSI) effects into spring-bed models of buildings on shallow foundations. Spring-bed models provide a balance between ease of implementation and theoretically rigorous solutions, as well as ability to include foundation uplift and non-linear soil deformation into earthquake analysis of multi-storey buildings on shallow foundations. The simple spring-bed model is best suited to shallow foundations that have a large static factor of safety against bearing capacity failure. In addition the modelling of shallow foundation nonlinear behaviour using a macro-element is discussed; this is particularly applicable to shallow foundations where the static bearing strength factor of safety is in the more usual 3 to 5 range. These two approaches to shallow foundation design are complementary.

For foundations using long piles it is explained how there are alternatives to the usual Winkler spring modelling of pile foundation lateral load behaviour.

Based on snap-back testing of near prototype scale shallow and pile foundations it appears that the question of damping still requires much work, although once nonlinear behaviour is engaged the amount of damping is often large.

The content of the paper is presented within the context of some of the recommendations of the Report of the Canterbury Earthquakes Royal Commission. It is proposed that the criterion for satisfactory foundation performance during the course of an earthquake is the residual deformation after the earthquake; suggestions for possible allowable residual deformations are made to encourage discussion of these limits. This is proposed as a more satisfactory approach than placing limitations on the proportion of the shallow foundation bearing strength that may be developed during the course of the earthquake. The Report of the Royal Commission has accepted this thinking for foundations using long piles.

The paper says nothing about the effects of liquefaction on shallow foundations. It is assumed that all future building designs will involve thorough consideration of the possibility of liquefaction and the consequences of lateral spreading. Thus shallow foundations will only be possible in dense material with good permeability or after extensive ground improvement has been undertaken.

1 INTRODUCTION

Nobody disputes the fact that a structure and the supporting foundation form a single entity. This paper reports on a programme of research work promoting the idea that the building design process needs to reflect closer interaction between the structural and geotechnical teams; an idea which the author has been promoting for more than ten years and which, recently, received support from the Canterbury Earthquakes Royal Commission, (2012) Final Report, Volume 1, Section 1: recommendation 53: "There should be greater cooperation and dialogue between geotechnical and structural engineers.". The motivation for this comes from the lessons learned from Christchurch. The observed performance of many structure-foundation systems makes clear that, in the past, the link between these two groups has been weak. For the future, the achievement of best practice requires more interaction, particularly in an environment where the soil conditions are variable and complex, that is for much of New Zealand.

The design of foundations to resist earthquake loading is a "very broad activity requiring the synthesis of insight, creativity, technical knowledge and experience" (Pender (1995)). Information is required and decisions have to be made at various stages including (Pecker and Pender (2000)):

a) the geological environment and geotechnical characterisation of the soil profile;
b) the investigation of possible solutions;
c) the definition of loads that will be applied to the foundation soil by the facility to be constructed;
d) information about the required performance of the structure;
e) the evaluation of load capacity, assessment of reserve of strength and estimates of deformations;
f) consideration of construction methods and constraints that need to be satisfied (finance and time);
g) exercise of judgement to assess potential risks.

Clearly the process outlined above is not a linear progression. All steps are closely interrelated and several iterations may be required, at least from steps (b) to (f).

The paper presents a summary of research done to date (which relate to items (c) and (e) in the above list), indicates future directions for the work, and considers the relevance of some of the recommendations in the Canterbury Earthquakes Royal Commission Report to the integrated design of structure-foundation systems.

The content of the paper is then promoting the idea encapsulated in Recommendation 3a of the Interim Report section 3.2 of the Royal Commission which states that designers of new buildings should:

> **"**Carry out in-depth analysis of the soil foundation super-structure system so as to ascertain the likely performance of the system."

In other words, the Royal Commission is promoting the recognition of the unity of the structure and foundation needs to be considered right from the commencement of the design process.

The initial step in foundation design must be to ensure a satisfactory performance under gravity load only. This means that the settlement of the foundation must be acceptable and also that there must be an adequate reserve of bearing strength available. This is the starting point for design of the foundation to respond to earthquake loading; furthermore it is suggested below that the static settlement of the foundation gives a deformation against which any post-earthquake residual deformations could be assessed.

2 FIELD TESTING ON SHALLOW FOUNDATIONS

Field experiments have been conducted at a site in Auckland with shallow foundations subject to

gradual pull-back followed by cyclic response after snap-back release; more details are given by Algie et al (2010) and Algie (2011). The set-up for the snap-back testing procedure is shown Figure 1. The ends of the steel frame shown in Figure 1 were supported on shallow foundations at each end; reinforced concrete 2.0 m in length and 0.4 m square. There were four sets of shallow foundations so the tests could be repeated at four different "sites". The steel frame structure was 2 m wide, 3.5 m high and 6 m long. Steel kentledge was strapped to the top of the frame to provide the required vertical foundation load.

The site used for the tests, in Albany in the northern part of Auckland, consists of a profile of stiff cohesive soil formed by in situ weathering from tertiary age sandstone and siltstone (it is thus a residual soil profile). The soil profile was investigated with 21 CPT tests between the surface and depth of 5 to 8 m; in some of these the shear wave velocity of the soil was measured which indicated a reasonably consistent shear wave velocity for the materials at the site equivalent to a small strain shear modulus for the soil of about 40 MPa.

The response of the system to one impulsive excitation is obtained with each snap release. An added bonus is the static load-deflection curve obtained during the pull-back phase of the test. A sequence of snaps from different initial loads shows how the nonlinear behaviour of the foundation develops as the applied load increases.

Figure 1. Shallow foundation set-up for snap back testing. Simple structure loaded with kentledge and supported on shallow foundations at the right. Chains attached to a hydraulic ram with a quick release shackle connected to the crane to provide reaction.

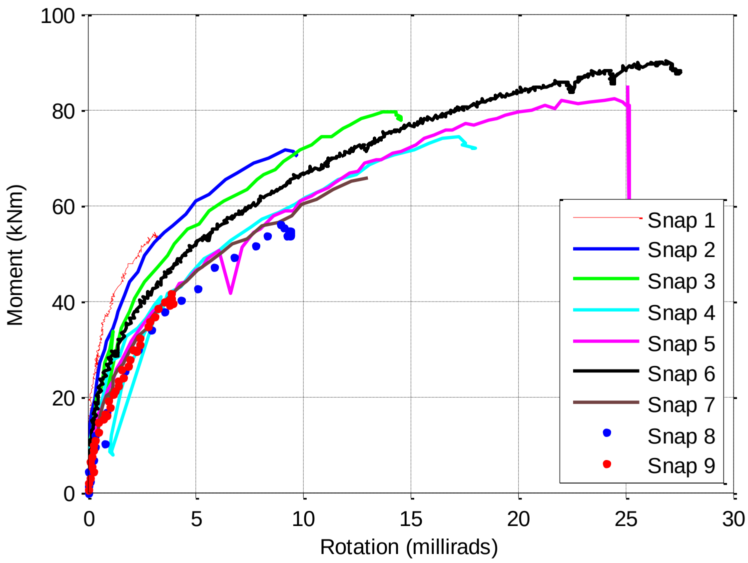

Figure 2. Pull-back data for the structure-foundation system shown in Fig. 1.

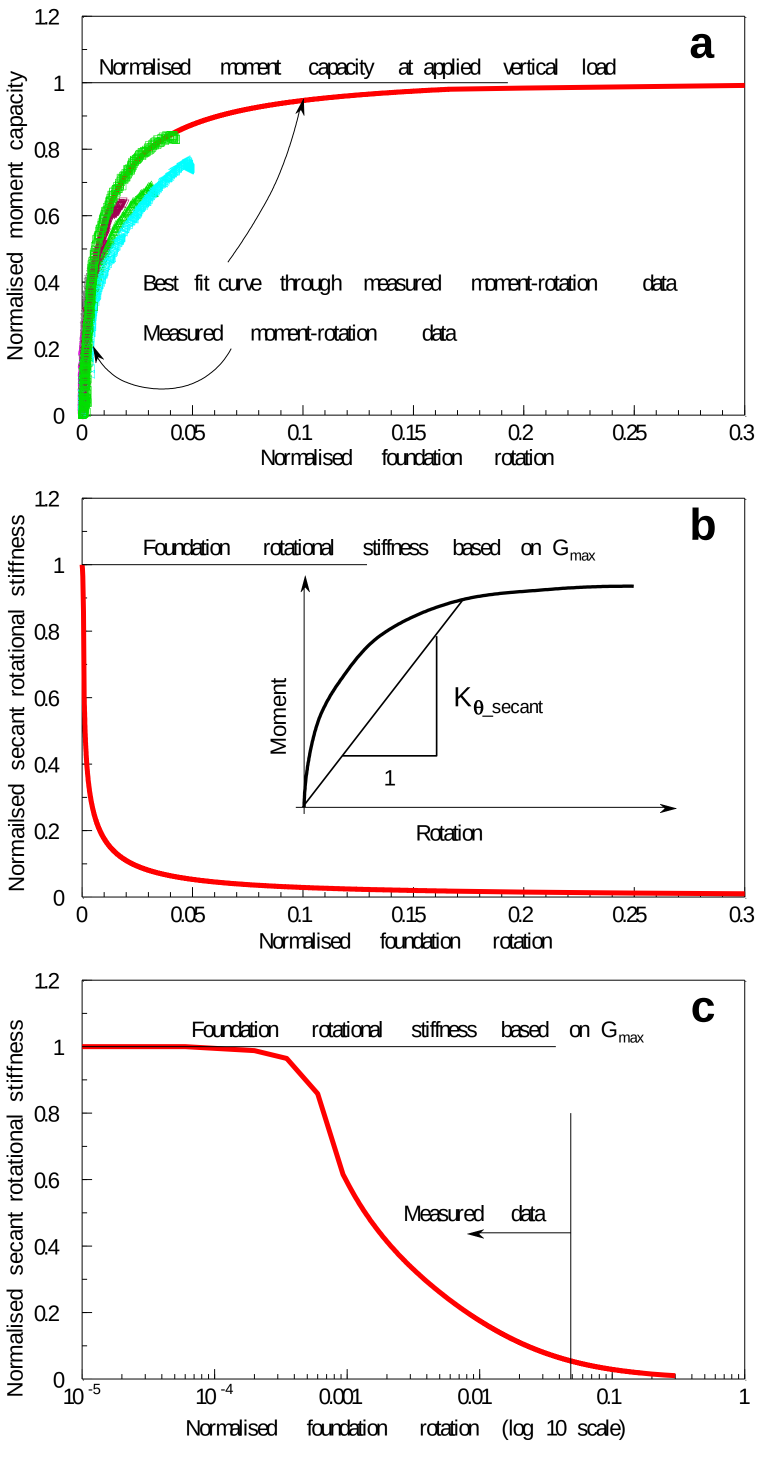

Figure 3. Curve-fitted moment-rotation relations matched to the recorded data for the first three pull-backs in Figure 2. (a) moment-rotation data, (b) and (c) secant modulus against foundation rotation.

In Figure 2 are shown all the static moment-rotation curves obtained during the application of the pull-back forces to one of the foundation sets at the site. It is apparent that there is considerable nonlinearity in the moment-rotation curves and also that the stiffness is reduced from one snap-back to the next, in particular for those tests following the snap-back which applies the largest moment to the system.

In Figure 3a the data presented in Figure 2 for the initial three (stiffest) pull-backs were used to define the foundation hyperbolic moment-rotation curve. The reason that the stiffest responses were used for the curve fit is because this will minimise any SFSI effect. (The observed degradation of the

subsequent foundation moment-rotation curves is, presumably, because of the irrecoverable deformation of the ground beneath the edges of the foundations.) Also shown in this figure is the estimated moment capacity of the foundation for the applied vertical load. The graph shows clearly that the hyperbolic curve fits the recorded data very well and that the moment capacity controls the curve for the extrapolation beyond the recorded data.

An important parameter in the development of the hyperbolic curve fit is the initial rotational stiffness of the foundation which can be obtained using the formulae of Gazetas (1991). Since this is the stiffness at very small load the small strain elastic modulus of the soil might seem to be the appropriate value to use. However, then the first part of the moment-rotation curve is far too stiff, so an "operational" stiffness, about one third to a quarter of the small strain value (EuroCode 8, CEN 2003), was used in calculating the hyperbolic curve in Figure 3.

In a famous paper, Housner (1963), a relationship was given between the period of a rigid block rocking on a rigid surface and the initial angle of rotation. The early part of Housner's curve is plotted in Figure 4 (the parameter α on the horizontal axis is the angle of tilt at which the block would fall over, for the structure shown in Figure 1 this angle is about 0.3 radians). Also plotted in Figure 4 are the half periods measured during the rocking response in the snap-back tests which are seen to fall along the Housner curve.

Interpretation of the test data in terms of conventional ideas about equivalent viscous damping were not satisfactory, there was much scatter in the data. Similarly looking at the damping from the perspective of Coulomb damping did not give good results. However, a reasonable match was obtained using the damping idea Housner presented in his paper where there is a loss in energy at each change in direction of the rocking motion. The data shown in Fig. 5 are seen to be matched well with the Housner parameter of 0.4. This suggests that there is relatively constant damping (ie energy loss with each reversal of the direction of motion).

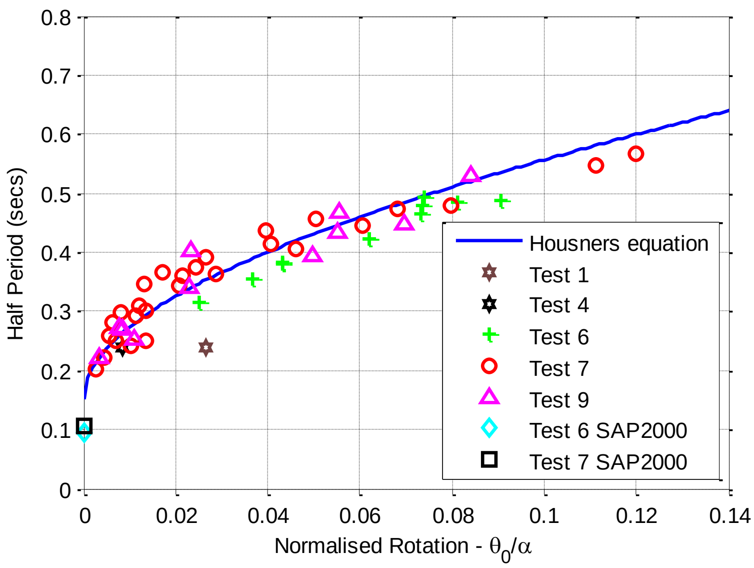

Figure 4. Section of Housner's rocking period function, with measured periods from all the snap-back tests, plus fixed base periods from the SAP2000 analysis (after Algie (2011)).

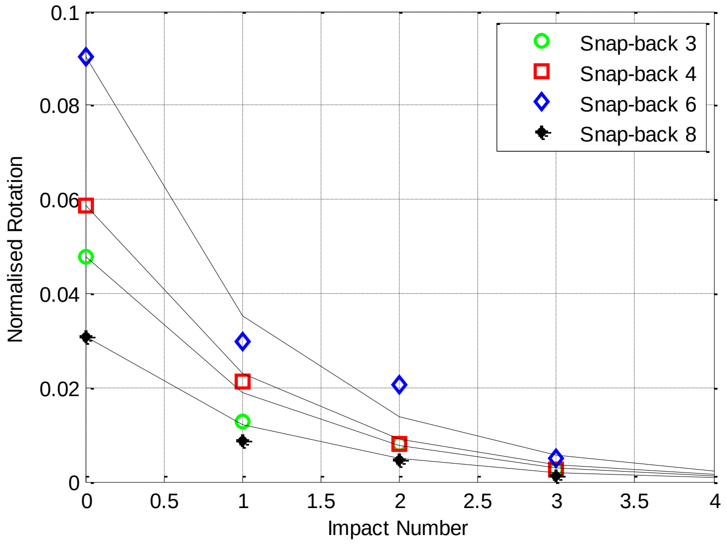

Figure 5. Normalised rotation against the number of reversals, matched with the Housner damping factor value of 0.4 (after Algie (2011)).

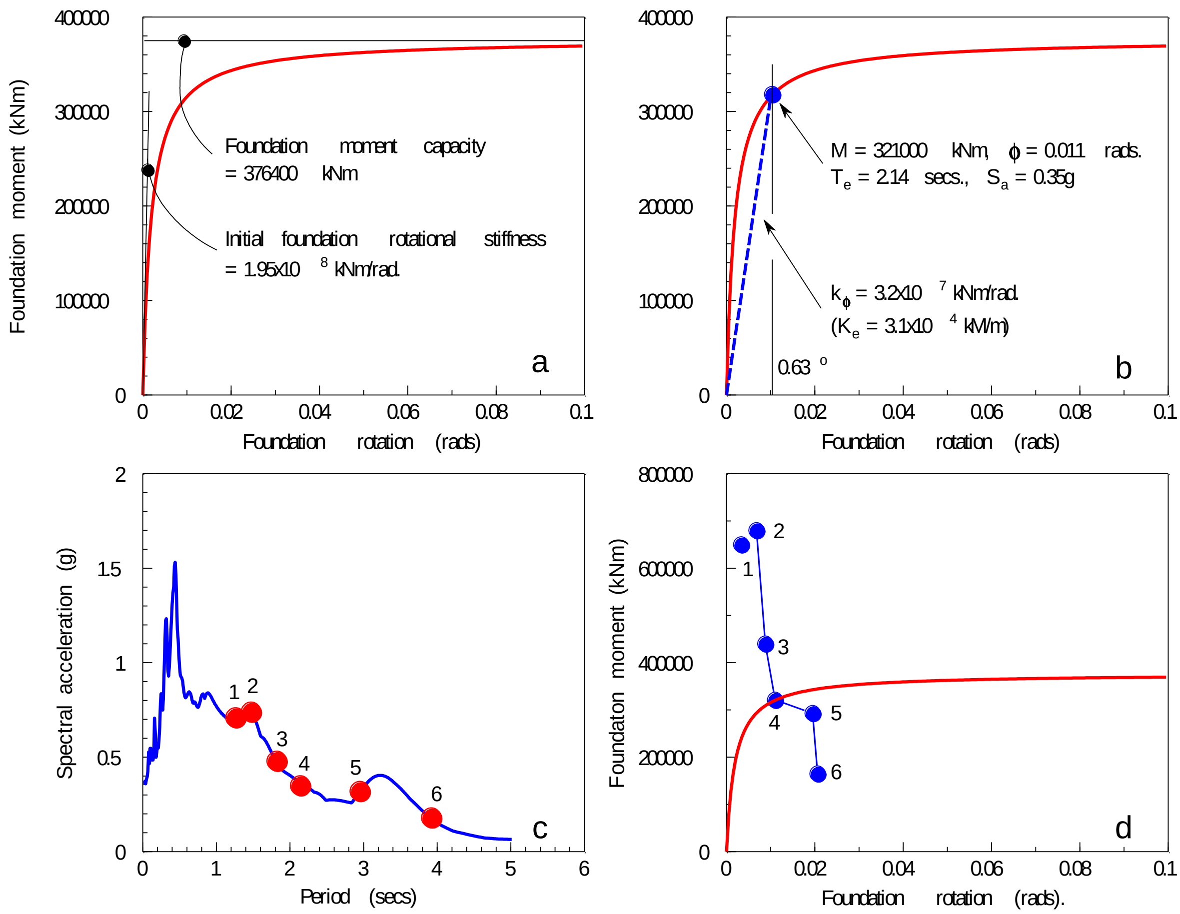

Figure 6. "Hand" method of calculation of system response: (a) properties of the structure-foundation system, (b) details of the response where the earthquake demand matches the foundation behaviour, (c) ground motion response spectrum showing the various trials, (d) moment-rotation data for the various trials in relation to the foundation moment-rotation curve. (after Pender et al (2013)).

3 SHALLOW FOUNDATION SPRING BED MODELLING

Two approaches are discussed below. First, a "hands-on" approach enabling a simple iterative design calculation, Pender et al (2013). Second, the modelling facilities of the commercial software, SAP2000, are utilised, Storie and Pender (2013 and 2014). Each of these is capable of including nonlinear soil-foundation-structure interaction (SFSI) effects into shallow foundation design analysis. Spring-bed models provide a balance between ease of implementation and theoretically rigorous solutions, as well as ability to include foundation uplift and non-linear soil deformation into earthquake analysis of multi-storey buildings on shallow foundations. The simple spring-bed model is best suited to shallow foundations that have a large static factor of safety against bearing capacity failure.

Kelly (2009) made proposals for the design of shallow foundations that rock during earthquake excitation. Rocking is understood as cyclic uplifting and reattachment at the ends of the foundation during the course of the earthquake excitation. Kelly explained that low to medium-rise structures on shallow foundations may not have sufficient weight to prevent foundation rocking; in which case the designer might want to take advantage of the real benefits that follow from accepting modest amounts of rocking. One of the topics Kelly recommended for further research was an appropriate method of handling soil nonlinearity; an approach to this is given below.

Field data gathered on the rocking response of shallow foundations on Auckland residual clay was reported and analysed by Algie et al (2010) and Algie (2011). This work, and related finite element modelling, provides the basis of a method of determining nonlinear moment-rotation curves for shallow foundations with uplift. Also required is information about the hysteretic damping associated with foundation rocking. Abaqus (Simulia 2010) and Plaxis 3D (Plaxis 2012) have been used to obtain this hysteretic damping information. Finally, the substitute structure method of Shibata and Sozen (1976), utilised recently by Priestley et al. (2007), is used to obtain a single degree of freedom model of the structure - foundation system. One additional aspect needs to be included in the substitute structure model, the compliance of the soil beneath and adjacent to the foundation. This is done by representing the structure-foundation system as a SDOF structure supported on a rotational spring and a horizontal spring.

With these tools a "hand" calculation method is available for estimating the earthquake response of shallow foundations which may uplift or rock. The calculation process is an iterative one but relatively simple; a set of results is given in Fig. 6. This calculation method can be considered to be an extension of the spring-bed modelling discussed by Taylor et al (1981).

The process outlined above is attractive because of its simplicity and "hands-on" calculation. However, other approaches are possible. Commercial structural analysis software suites provide an option to represent the foundation as a bed of springs, and some have springs that can detach and reattach. An example, using detachable springs, is given in a companion paper by Storie and Pender (2013, 2014). Spring-bed models, with uniformly distributed springs of equal stiffness, do not give the correct rotational stiffness for a shallow foundation if the spring stiffnesses are assigned to match correctly the vertical stiffness of the foundation. To overcome this problem the FEMA 356 document (Federal Emergency Management Agency 2000) recommends placing stiffer springs at the edges of the foundation whilst keeping the overall foundation vertical stiffness the same. Figure 7 shows the hyperbolic moment-rotation curve developed herein in relation to push-over curves calculated with the SAP2000 software (CSI 2011) for the two spring-bed foundation models. This shows that the uniform spring-bed model matches very closely the hyperbolic relationship shown in Fig. 3. On the other hand, the FEMA 356 spring-bed over-predicts the foundation moment for rotations less that about 0.03 radians. This mismatch occurs at rotations which are important for the evaluation of SFSI effects on shallow foundation design.

An aspect of the above that requires further work is value of damping to be used in the design process. As shown in Fig. 5 field testing of shallow foundations indicates that classical damping models are not relevant. The same conclusion is reached below with regard to pile foundations (Figs. 19 and 20).

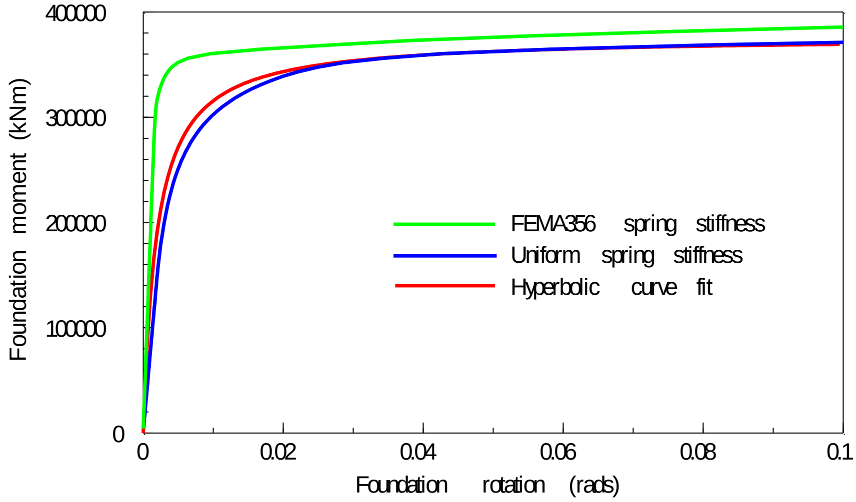

Figure 7. Comparison of the spring bed and the hyperbolic foundation moment-rotation curves.

To investigate the potential influence of SFSI in the earthquake performance of multi-story buildings on shallow foundations, generic 5, 10 and 15 story buildings were modelled. A number of assumptions were made about the size of the buildings, floor loading, and other properties to represent buildings typical to that found in the Christchurch CBD where shallow foundation performance appears to have been satisfactory following the Christchurch Earthquake. Equivalent SDOF models of these buildings were developed and are shown in Fig. 8. The procedures outlined by Priestley et al. (2007) were used, where a characteristic displacement defined an equivalent mass to be lumped at an equivalent height above the foundation. The stiffness of the column supporting the mass in the SDOF model was calculated using an assumed fixed base natural period (T_s) of the structure and this enabled an equivalent column size to be determined. For all the buildings, a 16 metre wide by 32 metre long raft foundation at the ground surface was modeled and a bed of 17 vertical springs captured the interaction between the foundation and the underlying soil for two dimensional analyses in the width direction.

The static elastic vertical stiffness of this foundation was determined using procedures outlined by Gazetas et al. (1985). This static stiffness value has the potential to be influenced by dynamic excitation and so further work by Gazetas (1991) was used to determine a dynamic elastic stiffness value. This was then distributed to the 17 vertical springs uniformly, based on each springs tributary area of the foundation. A uniform spring distribution was used because previous work by the authors has suggested it gives a close match to theoretical moment-rotation response derived from field testing, Fig. 7.

Horizontal stiffness of the foundation was assigned to a single horizontal spring, as shown in Fig. 1. The stiffness of this spring was calculated in a similar method to that for the vertical springs but follows the formulas developed by Gazetas and Tassoulas (1987). The horizontal dynamic factor from Gazetas (1991) was used to determine the dynamic elastic horizontal stiffness value used for the individual spring, and this spring was defined in the models as linear elastic.

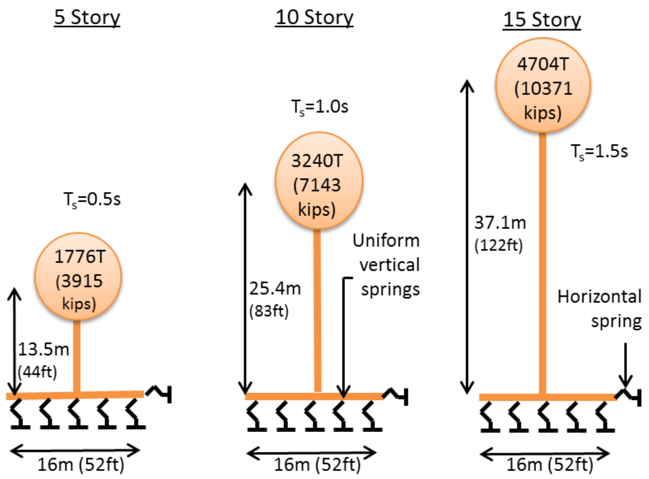

Figure 8. Details of SAP2000 SDOF models of the multi-story buildings

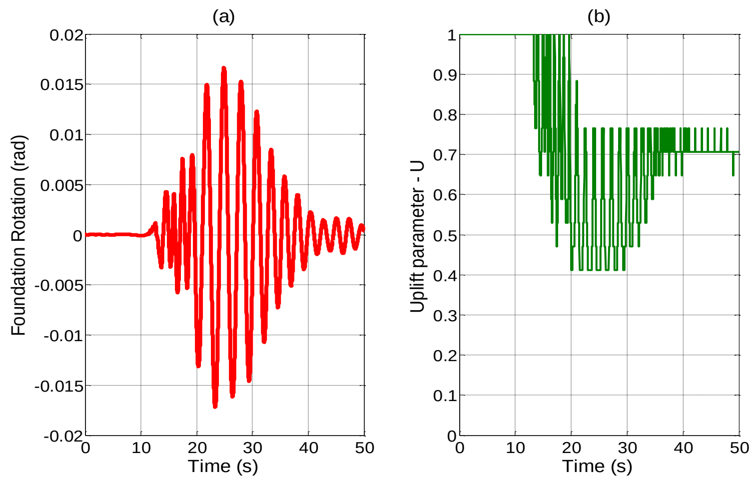

Figure 9. 10 story building earthquake time history response (CHHC record) of (a) foundation rotation (note the self-centring tendancy) and (b) uplift parameter *U* (springs attached / number of springs).

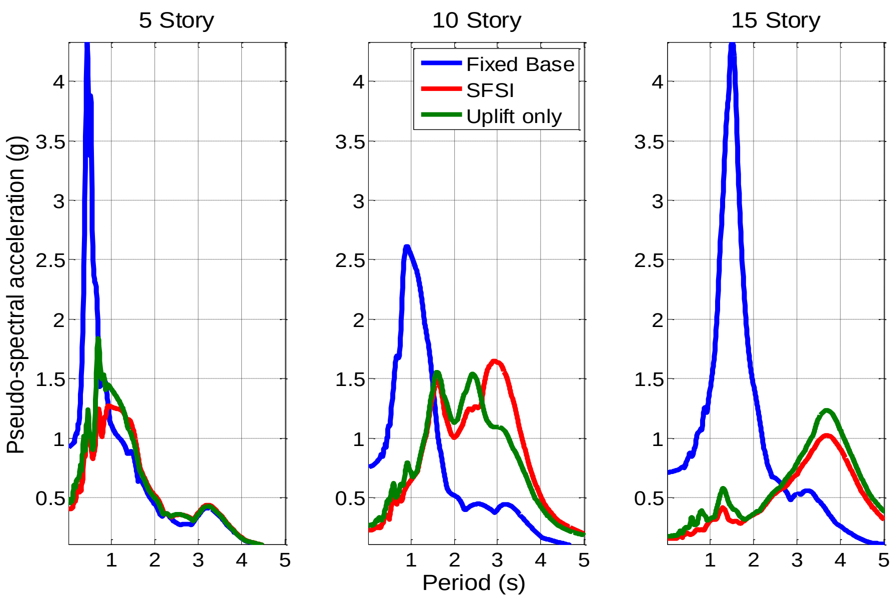

Figure 10. Pseudo-spectral acceleration response spectrum plots of the acceleration of the lumped mass of each building subjected to the CHHC record for fixed base, SSI, SFSI and uplift only conditions.

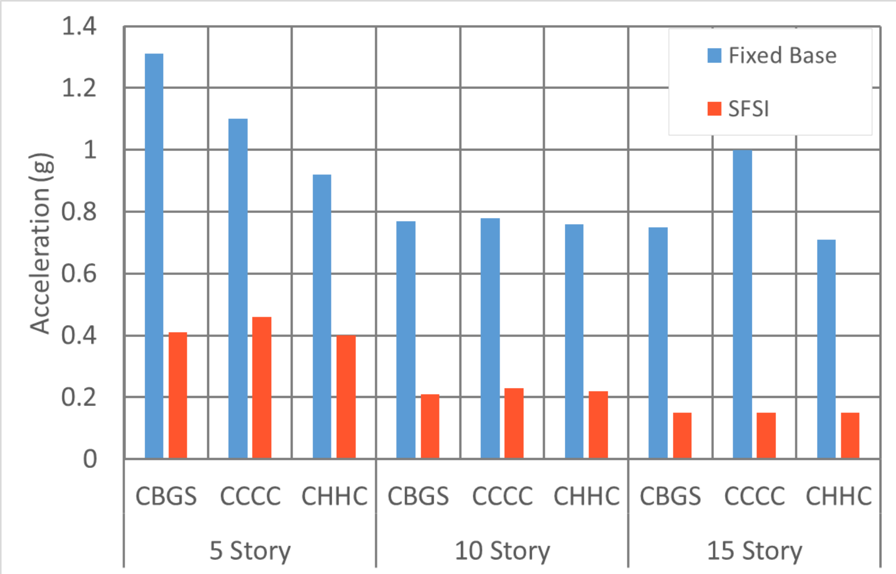

Figure 11. Peak accelerations of the lumped mass of the 5, 10 and 15 story SDOF models with fixed base and SFSI conditions subjected to 3 Christchurch Earthquake CBD records used in this study (CBGS, CCCC, and CHHC).

Figures 9, 10 and 11 show results from the modelling with SAP2000 using detachable and re-attachable springs. Figure 9 shows the tendency of the system to self-centre after the earthquake excitation and also indicates that after the passage of the earthquake that some the springs at the sides of the foundation are permanently detached. This is in-keeping with the spring bed models developed by Gajan et al (2011). Figure 10 shows the large effect that the detaching springs have on the response spectrum of the SDOF mass. The figure also shows that nonlinear deformation is not a significant influence on the response of the system, the main effect is the changes in the foundation stiffness that are induced by the detaching and reattaching of the springs. In other words, the behaviour of the system is highly nonlinear, but the nonlinearity is a geometrical effect rather than a consequence of soil nonlinearity. The same comment is applicable to the results displayed in Fig. 6. Finally, Fig.11 shows in another way just how significant is the spring detachment on the behaviour of the system.

The initial University of Auckland work at spring-bed modelling of foundation behaviour, both shallow and deep foundations, was done using the Ruaumoko software (Carr (2004)), Wotherspoon (2009) and Wotherspoon and Pender (2010).

4 MACRO-ELEMENT MODELLING OF SHALLOW FOUNDATIONS

The content of this section needs to be viewed in the light of Recommendation 16 of the Final Report Volume 1 of the Canterbury Earthquakes Royal Commission report:

> "For shallow foundations, soil yielding should be avoided under lateral loading by applying appropriate strength-reduction factors."

The word "should" is very significant as there has been underway for a couple of decades a strongly focussed body of research promoting just what this recommendation advises against (Paolucci (1997), Cremer et al (2001), Pender (2007), Gajan and Kutter (2007), Chatzigogos et al (2007), Anastasopoulos et al (2010), Paolucci et al (2011), Gazetas (2013)). The use of appropriate strength reduction factors in effect requires that the mobilisation of foundation bearing strength should never exceed more than about 50% of what is available, which would restrict the foundation to linear or near-linear behaviour so forgoing any benefits from the softening that occurs as the foundation bearing strength is approached.

However, Recommendation 12 of the Final Report Volume 1 of the Canterbury Earthquakes Royal Commission report states:

> "Foundation deformations should be assessed for the ULS load cases and overstrength actions, not just foundation strength (capacity). Deformations should not add unduly to the ductility demand of the structure or prevent the intended structural response."

This is in-line with what Pecker, Gazetas, Kutter, Paolucci and others have been promoting. Their thinking is that during the course of the earthquake it may not be of significance if the foundation bearing strength is mobilised for a brief instance or a few brief instances, what matters is the residual deformation after the event.

Recommendation 20 of Volume 1 of the Final Report of the Canterbury Earthquakes Royal Commission Report states:

> "Shallow foundations should be designed to resist the maximum design base shear of the building, so as to prevent sliding. Strength- reduction factors should be used."

It is not clear what the Commissioners had in mind in making this comment, because accompanying the base shear there is the foundation moment and the vertical load. What is important is that these actions be treated as a combination and to have an appreciation of where the net effect of vertical load, moment, and shear take the foundation in relation to bearing strength. However, bearing strength is not a simple number (despite the impression created by elementary foundation engineering texts) but a three dimensional surface with axes of vertical load, horizontal shear, and moment - when the combination of these falls on the bearing strength surface then we have a bearing strength failure, which means that unlimited foundation deformation will occur if the actions are maintained, but only some small residual deformation if the actions are maintained for only a brief period of time.

A model that uses these ideas is to represent the soil-foundation interaction by means of a so-called macro-element, shown in Fig. 12. The essence of the macro-element is to recognise that shallow foundation bearing strength is not a single number, and to use the bearing strength surface as a yield surface, a consequence of this is that any nonlinear soil behaviour occurs near the foundation and the soil beyond behaves elastically. Quite sophisticated versions of the macro element have been developed by Cremer et al (2001) and Chatzigogos et al (2007). At Auckland a more modest version has been formulated, Toh (2010), Toh and Pender (2010) and Toh et al. (2011), which has elastic behaviour of the foundation for load paths inside the bearing strength surface and perfectly plastic behaviour when the yield locus (bearing strength surface) is engaged. Some of our findings are presented in Figs. 13 to 16 which present results for a structure representing a massive bridge pier on a shallow foundation. The earthquake time history input was El Centro 1940 scaled to give a range of peak ground accelerations.

Figure 13 makes clear that the maximum nonlinear deformation correlates well with the spectral acceleration at the natural period of the structure-foundation system and not with the PGA of incoming earthquake motion. Figure 14 is important because it shows the earthquake induced settlement, that is a component of the residual deformation of the foundation, is a small fraction of the static settlement of the foundation for spectral accelerations up to about 0.5g. The information in Fig. 15 complements that in Fig. 14 and shows that residual rotations are small and less than about half the elastic rotation during the time history calculations. The maximum residual rotation is 5 milli-radians or less for earthquake spectral accelerations up to values in excess of 0.5 g (5 mrad. is a little less than 0.3 degrees). Figure 16 gives the results of a separate investigation where the effect of variability of soil properties and the variability of earthquake motions is investigated. The shallow foundation is founded on saturated clay and the undrained shear strength considered when the coefficient of variation ranged between 0.10 and 0.50 with a mean value of 100 kPa; the soil stiffness was taken as a multiple of the undrained shear strength. Also a suite of seven earthquake records, Oyarzo-Vera (2010), was used. Figure 16 indicates that the earthquake to earthquake variability had a greater effect on the results than the variability of the soil properties.

Recently a shallow foundation macro-element has been incorporated into the Ruaumoko software, Moghaddasi (2012).

A comparative study was done using centrifuge results from UC Davis of a foundation for a bridge

structure. The response of the system was calculated with the Toh macro-element model, a Ruaumoko bed of springs, and an OpenSees (Mazzoni et al (2009)) spring bed, Pender et al. (2009). The results indicated that all three approaches gave comparable results with small residual displacements, which, from the design perspective, is encouraging as it indicates that the most important step is to use some method of modelling foundation nonlinearity and this may be more significant than the actual details of how the nonlinearity is handled.

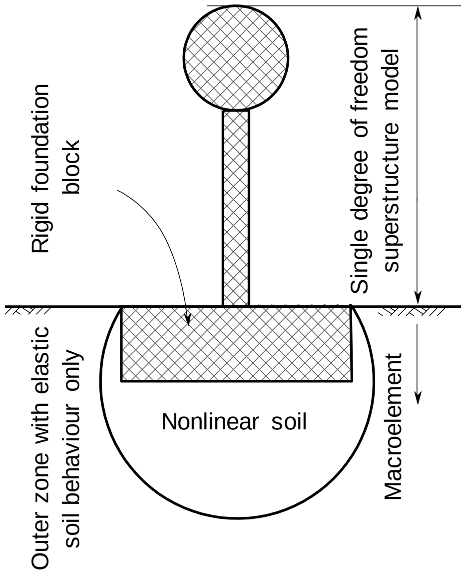

Figure 12. Macro-element concept (after Cremer et al, 2001).

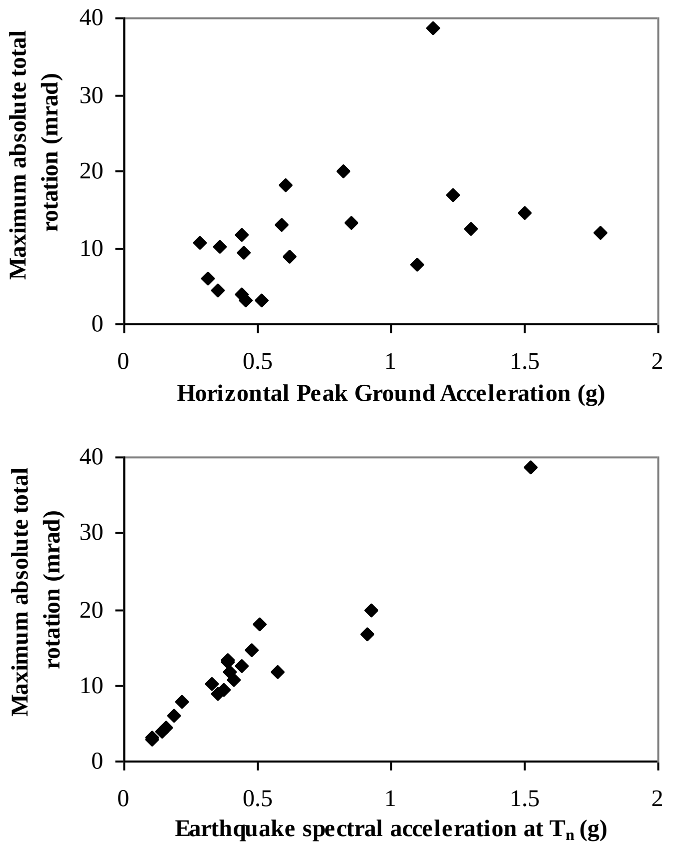

Figure 13. Residual foundation rotation versus PGA and spectral acceleration at system natural period.

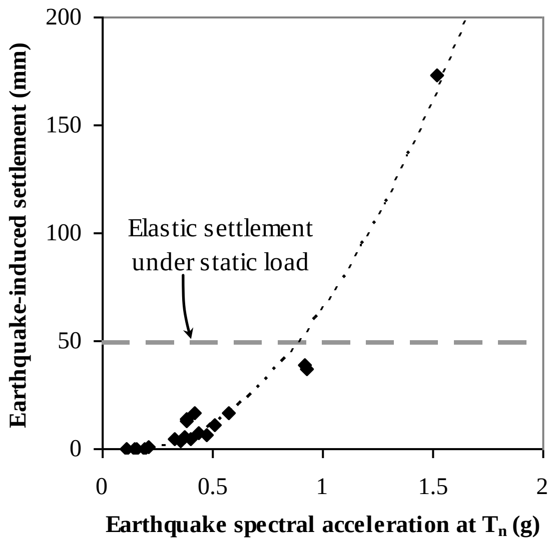

Figure 14. Earthquake-induced foundation settlement versus spectral acceleration at system natural period.

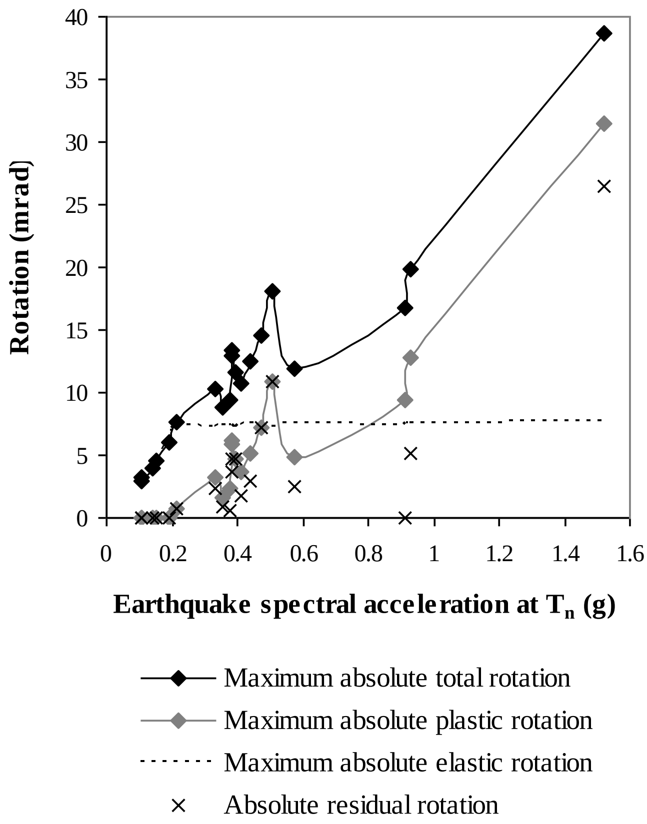

Figure 15. Elastic and plastic foundation rotation versus spectral acceleration at system natural period.

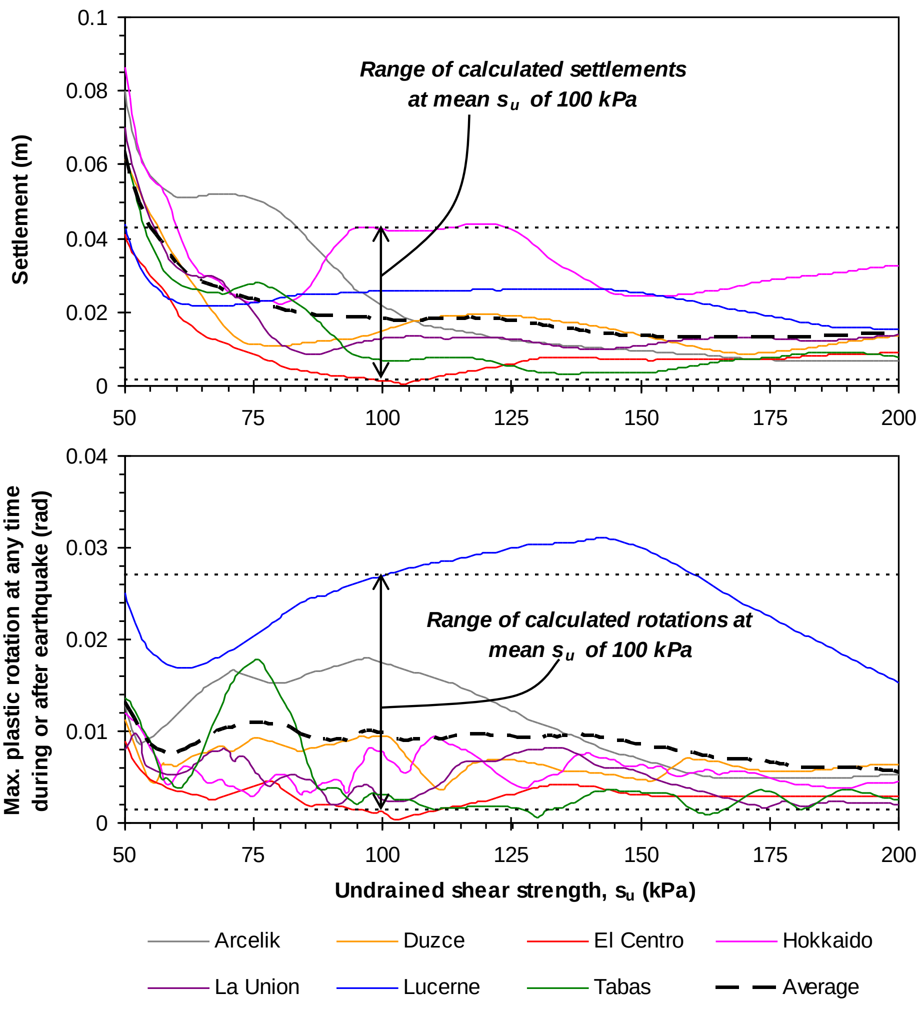

Figure 16. Calculated foundation displacements across the range of soil strength considered, for all 7 earthquakes.

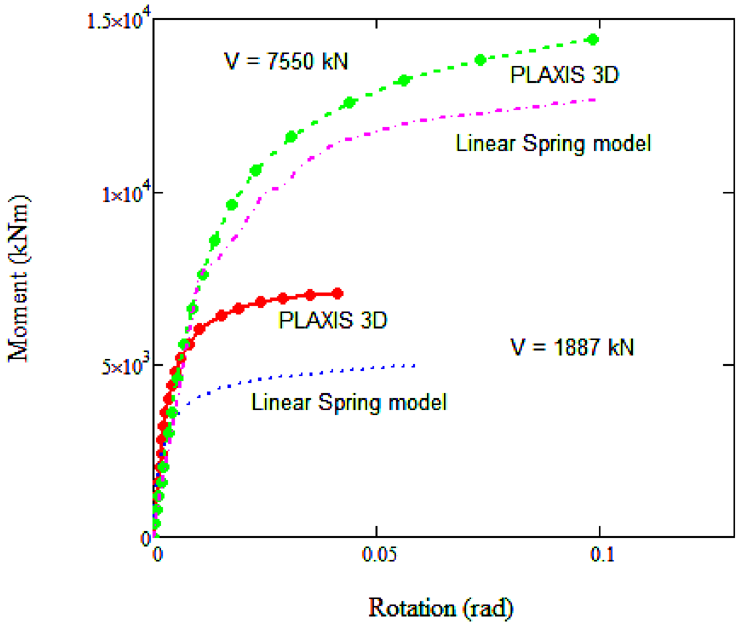

Figure 17. Moment rotation curves obtained from PLAXIS 3D analysis and linear spring model for 6mx6m footing for s_u = 85kPa case with constant vertical loads of V = 1887kN and V = 7550kN.

5 FINITE ELEMENT STUDIES OF SHALLOW FOUNDATION MOMENT-ROTATION RESPONSE

Neither the macro-element shallow foundation model nor the bed of nonlinear detachable springs is a complete model of shallow foundation behaviour, but they are known to be good approximations and useful design approaches. Nevertheless, it is instructive to mobilise the full rigour of three dimensional nonlinear finite element analysis using an interface between the foundation and the underlying soil that cannot not transmit tensile stresses to the soil. The results of some of this analysis using Plaxis 3D, (PLAXIS BV (2012)), are shown in Fig. 17. This shows that the response of the system depends on the vertical load carried by the foundation prior to the application of moment, as this load increases the two moment rotation curves compare better. Not shown in Fig. 17 are the contact pressure distributions between the foundation and the soil, the finite element analysis indicates that this is more complex than the pressure distribution given by the bed of springs model; more details are given by Salimath and Pender (2014).

6 FIELD TESTING OF PILE FOUNDATIONS

Pile lateral load tests, both static and dynamic, were done at a site in the Pine Hill subdivision at Albany, near Auckland. The material at the site was classified as Auckland residual clay, a product of the in-situ weathering of Waitemata Group tertiary age sandstones and siltstones. The CPT recorded an average cone penetration resistance of about 1 MPa with friction ratio of 1.5% to 6%. Shear-wave velocities recorded from the seismic CPT was approximately 160 m/s and was fairly constant with depth. Undrained shear strength (s_u) values of about 100 kPa were estimated from the CPT results. In addition other in situ tests were done to determine the dynamic stiffness of the soils at the site using the WAK (wave-activated stiffness) and spectral analysis of surface waves (SASW) methods as well as seismic CPT testing. All of these methods lead to small strain shear moduli in the 30 to 40 MPa range. The pore pressure measurements from the CPT soundings indicate that the water table is at a depth of 5 m, however, in the winter rainy season in Auckland, and for much of the rest of the year, the fine grained residual soil is expected to be saturated to near the ground surface, Wesley (2010).

Four closed-ended steel pipe piles, having an outside diameter of 273 mm, wall thickness of 9.3 mm, and lengths of 7.5 m, were driven to depths of 6.5 to 7.0 m using a 3000 kg drop hammer. The yield moment of the pile section is approximately 180 kNm.

Static lateral loading was done on two of the piles by connecting a manually operated hydraulic jack between the pile shaft projecting above the ground surface and the crane or an adjacent pile; the test arrangement is shown in Fig. 18. The piles were subject to a number of load-unload cycles with increasing maximum loads, but the maximum value was such that the yield moment of the pile shaft was not reached. The piles were subject to dynamic loading with an eccentric mass shaking machine attached to the top of the pile shaft and also to snap-back testing. Figure 19 gives results of two-of these; it is notable how the response changes as the pull-back force increases. For small values of the force, 10 kN, the response is what would be expected of an elastic system or one with only mild nonlinearity. For the 40 kN pull back load there is a very rapid attenuation of the response within the first cycle; this suggests a large amount of damping. Figure 20 shows the time history of the response from four successive pull-backs from a force of 120 kN; these indicate that the response is very heavily damped and that there is little subsequent vibration. Clearly the snap-back response of these piles is far removed from what would be expected for an elastic viscously damped system.

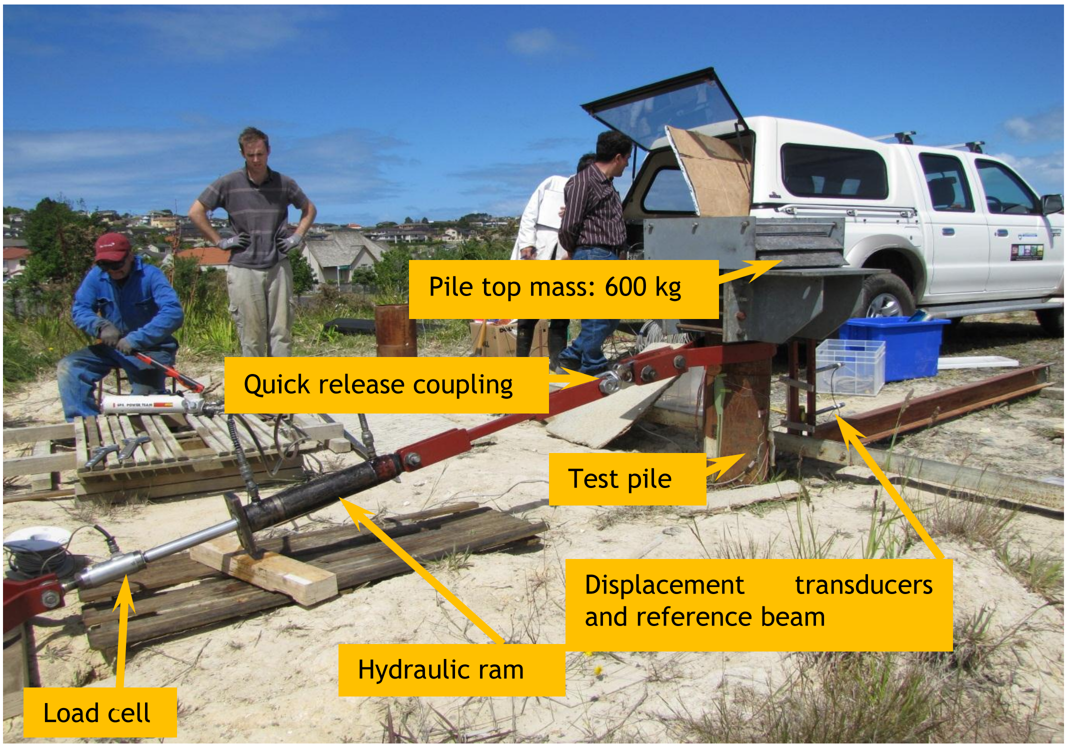

Figure 18. Field testing set-up for pile snap-back testing.

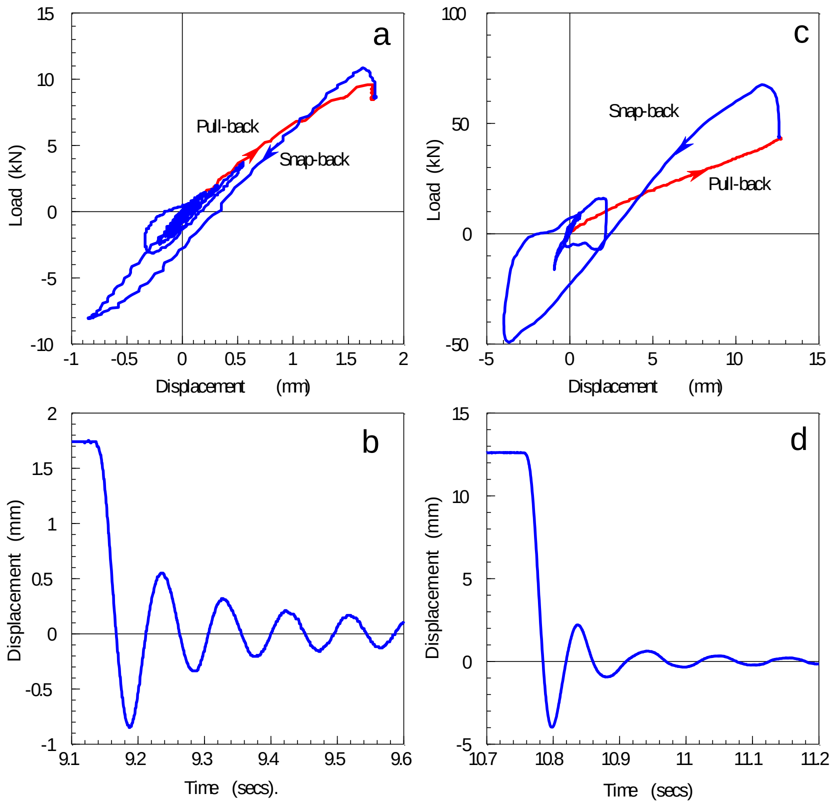

Figure 19. Pile lateral response data for two tests with pull-back forces of 10 kN (a & b) and 40 kN (c & d). (a) pull-back and snap-back load deformation response from a pull-back force of about 10 kN; (b) time – displacement response after release from the 10 kN pull-back; (c) pull-back and snap-back load deformation response from a pull-back force of about 40 kN; (d) time – displacement response after release from the 40 kN pull-back.

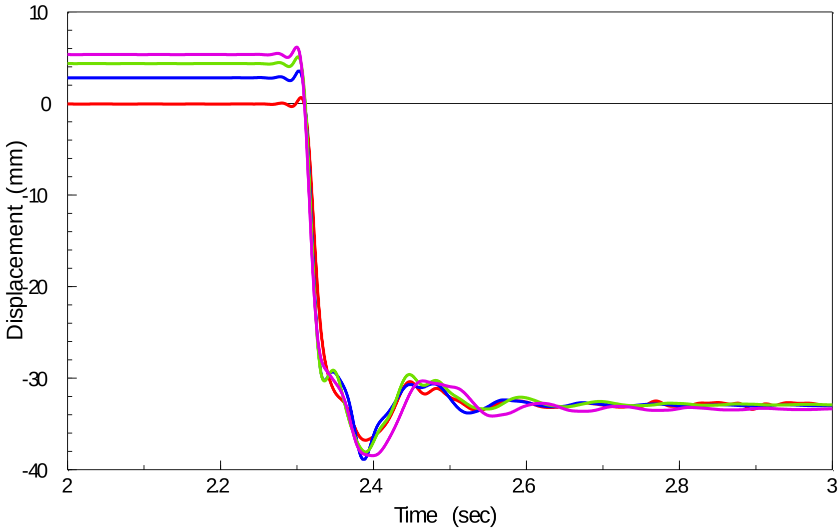

Figure 20. Snap-back response four releases from 120 kN pull-back

6 FINITE ELEMENT STUDIES OF PILE FOUNDATIONS

The load-unload curve for one of the piles up to a maximum lateral load to 125 kN is shown in Fig. 21. There are two calculated load-displacement curves on the diagram, one obtained with the finite element software OpenSeesPL, Lu et al (2010), and the other using the pile-head macro-element, Davies and Budhu (1986). To achieve the match in the plots it is necessary to use an elastic modulus for the soil considerably less than the 40 MPa determined for the small strain shear modulus at the site. The operational modulus used was 10 MPa; more details are given by Pender et al. (2012). Thus we conclude that the operational modulus of the soil is about one third to one quarter of the small strain modulus.

Finally, Fig. 22 shows hysteresis loops calculated with an extension of the Davies and Budhu pile head macro-element. It is notable that the macro-element loops correspond well with those obtained when an eccentric mass shaking machine was mounted on the pile head.

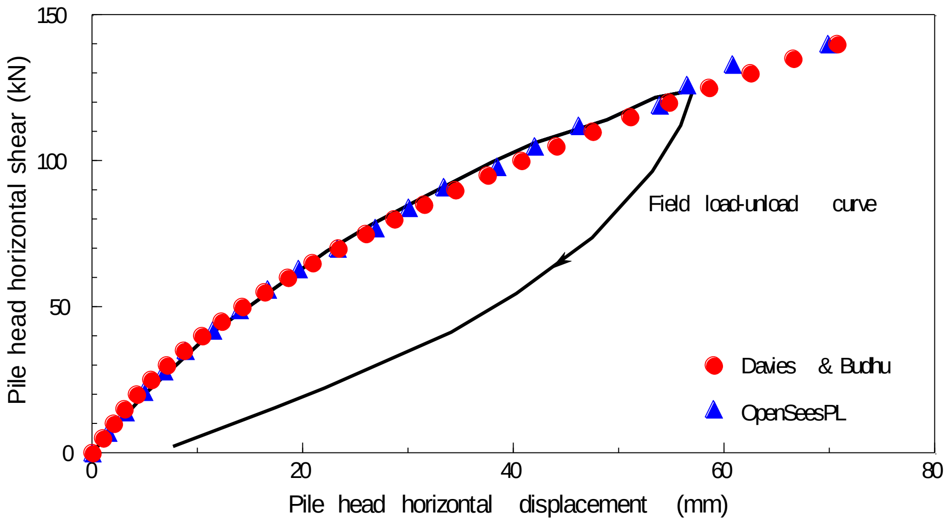

Figure 21. Comparison between the OpenSeesPL and Davies and Budhu predictions of the loading part of static load-deformation behaviour of the pile. (Soil undrained shear strength = 80 kPa and operational shear modulus = 10 MPa.)

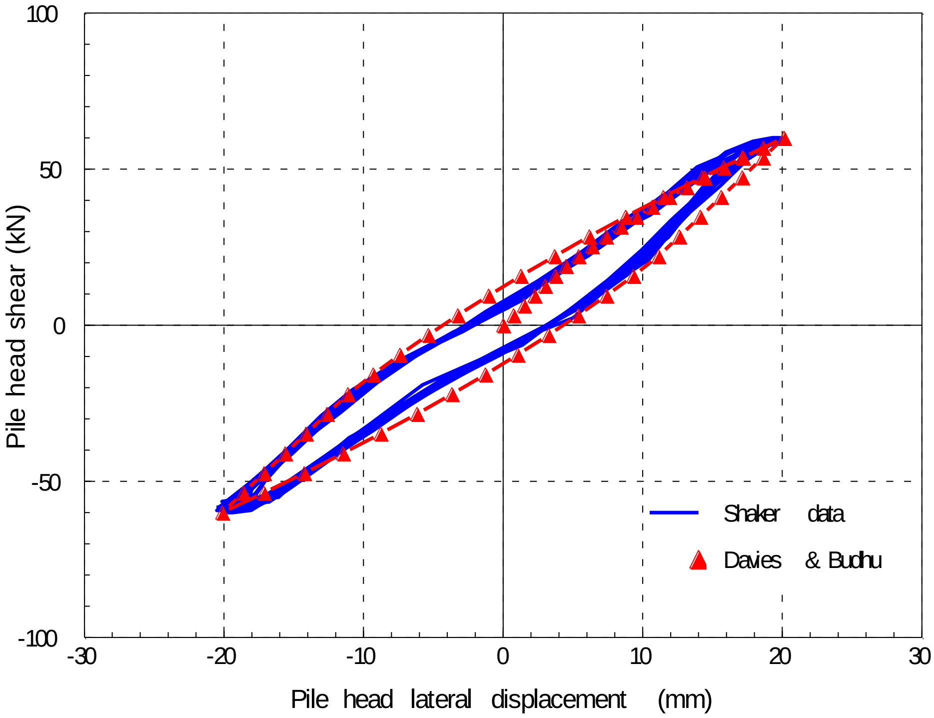

Figure 22. Comparison of the Budhu and Davies cyclic load deformation loops with those recorded during the dynamic excitation of the pile head. (Undrained shear strength = 80 kPa, soil shear modulus = 10 MPa, gap depth = 0.1 m)

Recommendation 17 of Volume 1 of the Final Report of the Canterbury Earthquakes Royal Commission Report states:

> For deep pile foundations, soil yielding should be permitted under lateral loading, provided that the piles have sufficient flexibility and ductility to accommodate the resulting displacements. In such cases, strength- reduction factors need not be applied.

This is an interesting contrast the statement about shallow foundation design, discussed above at the beginning of section 4 (Recommendation 16). My understanding is that for long piles the lateral capacity is controlled not by the soil strength but by the moment capacity of the pile section. In fact, for widely differing soil strength profiles there is little difference in the lateral capacity of the pile, although the depth to the maximum moment will be different. This provides a way of connecting Figs. 3 and 21. It is possible to re-plot the information in Fig. 21 in the form of Fig. 3. From here a similar "hands on" calculation process similar to that in Fig. 6 for a shallow foundation could be repeated for a pile foundation.

8 ALLOWABLE RESIDUAL DEFORMATIONS

So from what has been said above it should be clear that post earthquake residual deformation should be the design criterion.

Recommendation 13 of Volume 1 of the Final Report of the Canterbury Earthquakes Royal Commission Report states:

> "Guidelines for acceptable levels of foundation deformation for the ULS and overstrength load cases should be developed. The Department of Building and Housing should lead this process."

I have no wish to pre-empt the work of the Ministry of Business, Innovation and Enterprise, but perhaps some preliminary comments might help to get the discussion underway. There are three components of residual deformation: settlement, horizontal displacement, and tilting. Experience around Christchurch after the 2010 and 2011 events has shown that the unaided eye is easily able to detect tilting of a tall structure of about 0.5 degrees or more; so I suggest a limit on the acceptable post-earthquake tilt of 0.2 degrees (about 3 milli-radians). The static settlement of the building must have been estimated as part of the design process (even better has it been measured?), could we could tolerate additional settlement and residual horizontal displacement of 50% of this?

Then all connections to the building: telecommunications, electric power, water supply, gas, and sewer lines, need to be designed so that they can accommodation displacements of, say, twice these values.

How would these residual displacements be estimated? Using time history analysis either with a bed of springs foundation model or the macro-element model.

9 GEOTECHNICAL - STRUCTURAL COLLABORATION

The Canterbury Earthquakes Royal Commission makes the point in recommendation 53: "There should be greater cooperation and dialogue between geotechnical and structural engineers". It seems to me that this means much more than the cursory exchange of a few numbers and from then the geotechnical team has no role.

Two aspects of this that I think require particular attention. Recommendation 11 of Volume 1 of the Final Report of the Canterbury Earthquakes Royal Commission Report states:

> "Conservative assumptions should be made for soil parameters when assessing settlements for the SLS."

That is clearly the correct focus when considering the long term settlement of the foundation, as one wants to over-predict the settlement rather than under-predict it. The process, then, of achieving this prediction is to consider the values for the soil stiffness parameters and adopt a conservative assessment of the expected true mean value. However, this is not necessarily the correct approach when evaluating the stiffness for consideration of seismic response. If one uses values that have consistently been rounded down one has in effect a lower bound on the stiffness of the foundation and consequently the natural period of the structure foundation system will be at a maximum and so, if in the falling part of the response spectrum, then the foundation actions will be minimised. Consequently one also needs to consider what happens when the foundation stiffness is maximised, that is when the small strain shear modulus of the soil controls what happens. My personal approach to this has been to look at the response of the system when the foundation stiffness is controlled by the small strain shear modulus of the soil, and then do comparative calculations when the shear modulus is reduced by a factor of 4 or 5.

As mentioned above the bearing strength of a shallow foundation is not a single number. Thus the geotechnical and structural people need to work together to understand the nature of the actions applied to the foundation during the earthquake; frequently it will be the moment capacity of the foundation that is the controlling factor not the vertical load capacity.

Finally, a task that is the responsibility of the geotechnical team assisted engineering geological input is the development of a site model. This will also have an impact on how much site investigation work is done, or follow-up work is undertaken.

Recommendation 3 and 4 of Volume 1 of the Final Report of the Canterbury Earthquakes Royal Commission Report state:

> "A thorough and detailed geotechnical investigation of each building site, leading to development of a full site model, should be recognised as a key requirement for achieving good foundation performance."

> "There should be greater focus on geotechnical investigations to reduce the risk of unsatisfactory foundation performance. The Department of Building and Housing should lead the development of guidelines to ensure a more uniform standard for future investigations and as an aid to engineers and owners."

However, to achieve what is required by way of best practice more than investigation is necessary. Even the most elaborate site investigation can never obtain information about more than a tiny fraction of the ground which will be affected by the foundation to be constructed. Consequently, follow-up during the construction process is essential to confirm that what is exposed is consistent with the conclusions from the site investigation process and the assumptions on which the foundation system was designed.

10 SUMMARY AND REMAINING CHALLENGES

The major challenge going forward is how to integrate the contribution of the structural and geotechnical teams. At this stage I have no firm proposals, but it will require determined efforts of the two communities to develop completely new ways of working and communicating.

The results of field testing presented in this paper present much interesting information; clearly there is scope for more work of this kind. However, the big challenge at present is how to estimate damping values to be associated with nonlinear soil behaviour. Field testing and three dimensional nonlinear finite element analyses suggest that conventional equivalent viscous damping models are not relevant.

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