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Integrated Earthquake Resistant Design of Structure-Foundation Systems

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ABSTRACT

In this paper the case is made for an integrated approach to the earthquake resistant design of structure-foundation systems. Emphasis is placed on the need to analyse the response of a **system** that has the foundation and structure modelled with comparable levels of sophistication. The paper gives examples which illustrate what can be achieved with simplified models that represent the essence of the structural and foundation behaviour. However, to achieve a truly integrated structure-foundation design the investigation of the soil in which the foundation will be constructed needs to receive effort comparable to that expended in modelling the structure-foundation system. This requires accurate mapping of the soil types and layers present as well as estimation of the shear strength and stiffness of these materials. For the cyclic loading that occurs during an earthquake the shear stains in the soil near the foundation will be larger than those associated with shear wave propagation, so an "operational" stiffness is needed for the soil. Field test data for shallow and deep foundations at a site in Auckland residual soil are presented to show the extent of soil softening during foundation cyclic loading.

INTRODUCTION

Given the very powerful computer resources that are now available for civil engineering and infrastructure design, a pressing need is to improve interaction between the structural and geotechnical communities. An obvious priority is for the two groups to work together in a more integrated fashion. The author has been promoting this view for some years; the suggestion is universally greeted with assent. Recently, similar observations have been made with regard to the design of very tall buildings, Poulos (2009) and Baker (2010). In addition the FEMA 356 (Federal Emergency Management Agency (2000)) document also emphasizes that foundation and structural design need to be considered together (Section C.4.1).

The most direct way in which this can be achieved is by the two communities developing integrated numerical models of complete structure-foundation systems. Too often in the past the practice has been for the foundation and superstructure to be considered almost in isolation. Lapsing into anthropomorphism, one can say, that, from the perspective of an incoming earthquake, the structure and the foundation system supporting it appear as a single entity.

If this is accepted then the design approach needs to be based on a single integrated model of the building-foundation system. Nowadays exceedingly capable software is used for analysis and design of structures. The full potential of this software will not be realised until a complete model of the structure-foundation system is used. This point of view is certainly not based on the assumption that the future of engineering design lies in evermore sophisticated software, in a manner that reduces human input and minimises opportunities for engineering judgement. Rather it is intended that the exercise of engineering design judgement will be enhanced, so enabling the designer to obtain a more realistic understanding of the how the design will perform. This requires little new in the way of software facilities, what is needed is simply that the human side of the process is organised to realise the best output from the combination of numerical modeling and geotechnical evaluation of the materials present at a given site.

This integrated approach will be invaluable when applied to the design of new buildings and infrastructure. However, it may be even more valuable when applied to the assessment of existing foundations for facilities that are under consideration for retrofit. Here careful evaluation of the manner in which a foundation and the structure supported interact may lead to a better understanding of the actual capacity of the system and even, in some cases, the conclusion that retrofit is unnecessary. The reason that this could happen is that it is likely that such assessments will be more sophisticated than the original design and less dependent on a cascade of conservative decisions which has been a significant feature of foundation design in the past.

A well developed body of literature exists under the heading of Soil-Structure-Interaction (SSI). However, this is generally limited to consideration of contributions from soil beneath the foundation assumed to be elastic. An important point made herein is that the behaviour of the soil supporting and surrounding the foundation needs to be considered as extending into the nonlinear range. To emphasise this difference the term ***Soil-Foundation-Structure-Interaction*** (SFSI) is used in this paper.

SHALLOW FOUNDATIONS

Shallow foundations are, of course, feasible only in non-liquefiable soils. Liquefiable deposits will need either ground improvement to make them suitable for shallow foundations or, more likely, mandate the use of deep foundations. Likewise deposits of soft sedimentary clay, normally consolidated or lightly overconsolidated, will require deep foundations. This leaves sites with stiff cohesive soils as contenders for shallow foundations during earthquakes (although deep foundations could be used here also). Residual soils and overconsolidated sedimentary clays that would be described as stiff, or even hard, soils are thus candidates for shallow foundations.

The conventional wisdom is that shallow foundation design is controlled by static settlement. This statement is undoubtedly true when the long term behaviour of a foundation under static load is being considered, and particularly when one allows for the increase in bearing strength caused by the consolidation of the soil beneath from the stresses generated by the weight of the building and contents. Thus for larger foundations for which the static design is controlled by long term settlement considerations the reserve of static bearing strength under vertical load can be expected to be generous.

However, there are two important differences when it comes to earthquake loading. First shear and moment are applied to the foundation during cyclic loading, consequently the bearing strength is less than that under static vertical load. Second, since we are dealing with cohesive soils, the undrained shear strength will control the bearing strength of the foundation during the earthquake loading rather than the drained bearing strength which controls the long term static bearing strength.

Traditionally foundation bearing strength, in relation to the applied actions, has been expressed in terms of a bearing strength factor of safety; this lumps into one factor uncertainties associated with both the loading and the properties of the soil. More recent approaches have separate factors for foundation actions and soil properties – the so-called limit state design methods. These come in two styles - Load and Resistance Factored Design (LRFD) used in New Zealand, Canada and parts of the United States and the Partial Factor approach of European origin. A typical LRFD approach requires that under the design earthquake the demand on the foundation bearing strength should not exceed a certain fraction, say 50% to 75%, of the bearing strength. In New Zealand LRFD Design is used for proportioning shallow foundations under earthquake loading and the mobilisation of bearing strength is limited to about 50 to 60%, NZS1170.5 (Standards NZ (2004) and B1/VM4 (Department of Building and Housing (2003)). In Europe where a partial factor approach is used, the earthquake loading of a shallow foundation is restricted to mobilising about 75% of the foundation bearing strength.

Conventional SSI leads to the conclusion that for many structures the inclusion of foundation compliance effects will reduce the earthquake design actions imposed on the structure. The concept is that if the period of the structure is on the falling branch of the response spectrum then the period of the structure-foundation system will be longer than that for the fixed base structure and so reduce the earthquake actions (although the opposite will be true for a rising branch of the response spectrum). However, if one couples this thinking with the requirement that the bearing strength demand on the foundation must satisfy the LRFD requirement, then the foundation size may be limited by bearing strength considerations. This may require for taller structures that the plan area of the foundation needs to be larger than the plan area of the building. As the size of the foundation increases the rotational stiffness increases rapidly and this reduces the amount of period lengthening induced by the SSI effects, Pender and Butterworth (2003).

The message of the above paragraph is that driven by LRFD requirements the shallow foundation size may be such that any advantage in system performance from soil compliance may be offset by the large foundation required. One can ask, however, if brief instances of shallow foundation bearing strength failure during the course of an earthquake are actually unacceptable. The consequence will be some permanent displacement at the end of the earthquake. Perhaps the permanent displacement could become the foundation design criterion in just the same way as settlement is the controlling factor in shallow foundation design for static loads.

This suggestion is hardly radical, as this idea is well established in considering the earthquake response of earth dams and slopes (Newmark 1965) and gravity retaining structures (Richards & Elms 1979) in terms of the residual, or permanent, displacement at the end of the earthquake. In taking this approach one admits that brief instances of failure of the system during the course of the earthquake might not be important if the permanent displacements generated during the earthquake are modest. Perhaps one needs to extend the focus on residual displacement to the amplitudes of the cyclic displacements during the earthquake as factors, for tall structures, such as interaction with adjacent structures or damage to service connections. Even taking this extended view the question still remains as to whether brief instances of bearing strength failure during the course of an earthquake is a serious matter.

The design philosophy proposed above is not new, similar ideas have been discussed by Taylor et al (1981), Paolucci (1997), Cremer et al (2001), Gajan et al (2005), Ugalde et al (2007), Algie et al (2008), Anastasopoulos (2009), Pender et al (2009) and Toh and Pender (2009). All of these papers reach the conclusion that brief instances of bearing strength failure enhance the performance of shallow foundations in that they reduce the earthquake actions at the cost of modest residual deformations.

Example – tower structure on a block foundation

To demonstrate that brief instances of bearing failure during cyclic loading may not compromise a shallow foundation, this example compares the results obtained from dynamic tests on a shallow foundation system in the UC Davis centrifuge with numerical calculation of the response. The foundation system is represented using three different macro-element models.

The elastic structure rests on shallow foundations on a layer of clay consolidated from reconstituted San Francisco Bay Mud with an undrained shear strength of 100 kPa. The prototype dimensions of the footing are 2.67 m in length and 0.63 m in width. The effective height of the mass during dynamic excitation is 4.66 m. With these footing dimensions the static bearing strength factor of safety is 2.8. The dynamic input applied in the centrifuge was a ramped cosine wave of frequency about 1 Hertz and building to a maximum acceleration of 0.7 g over a duration of about 8 seconds. Further details of the centrifuge test and results are presented by Pender et al (2009) and Rosebrook and Kutter (2001). Figure 4 gives the measured response history of the centrifuge model in terms of moment - rotation, settlement - rotation and horizontal shear force - horizontal displacement.

Below the three different elements used to models the shallow foundation stiffness and strength are described briefly; a feature of all three models is the coupling between shear, moment and vertical loads. The elastic stiffness and radiation damping of the foundations was calculated using the expressions given by Gazetas (1991).

Foundation model 1: Bearing strength surface macro-element model. This is a version of the macro element of Paolucci (1997). A more sophisticated variant is that of Cremer et al

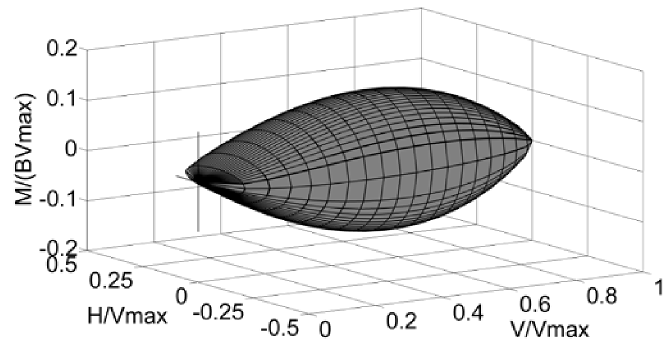

Figure 1 Normalised EC8 bearing strength surface for cohesive soil. (V – vertical load, H – horizontal shear, M – moment, B – foundation width, V_{max} – bearing strength for vertical load only.)

(2001). This model is based on a bearing strength surface which defines the combinations of vertical load, horizontal shear, and moment that cause bearing failure beneath a shallow foundation. Herein the surface defined in Eurocode 8 for cohesive soils is used; it is shown in Figure 1. The surface shows how the amount of moment and shear that can be applied to a shallow foundation depends on the vertical load. This surface acts as a yield locus, state paths inside the surface are elastic and those on the surface perfectly plastic, a non-associated plastic potential must also be specified. Elastic behaviour of the foundation is given by three springs the stiffness and associated radiation damping or which are calculated using the formulae of Gazetas (1991). Further details of the model are given by Toh (2008) and Pender et al (2009) and Toh and Pender (2009).

Foundation model 2: Spring-bed model in the Ruaumoko software. The Ruaumoko software, Carr (2003), is capable of dynamic time history nonlinear structural analysis. One of the elements provided is a nonlinear, compression only, detachable-reattachable spring. A bed of these springs provides the shallow foundation model which has the facility to uplift part of the foundation during cyclic loading and to reattach it at some stage after the direction of motion is reversed. The springs are bilinear so yielding is possible when the contact pressure reaches a limiting value. The details are shown in Figure 2. Radiation damping is included with values calculated from the Gazetas expressions.

Foundation model 3: Spring-bed model in the OpenSees software. This spring-bed model was developed in the OpenSees software (PEER 2009) and is very similar to the bed of springs model developed in Ruaumoko, the difference being that the springs in this model are non-linear, rather than bilinear as in Ruaumoko. Forty eight q-z springs were used in the vertical direction and one t-z spring in the horizontal direction. These non-linear spring models, QzSimple1 and TzSimple1, were developed by Boulanger (2000). Backbone curves for these springs have a hyperbolic shape and hysteretic damping is generated when the direction of loading is reversed. The FEMA 356 document suggests that spring-bed models should have the springs concentrated towards the outer ends of the foundation as shown in Figure 3; this was how the springs were arranged for this model. The tension capacity of the springs was set to zero so uplift of parts of the footing could be included.

As the three sets of calculations were done with different software, before proceeding to the nonlinear calculations a check was made on the results obtained when the foundation was modelled with linear elastic springs – one for the vertical stiffness, one for the horizontal stiffness, and one for the rotational stiffness. The stiffness values for these were estimated using the relations given by Gazetas (1991). The Young's modulus value used in estimating the stiffness was taken as $500s_u$, that is 50 MPa. The damping values were also calculated from Gazetas (1991). Once it was established that all three software packages gave the same elastic output the nonlinear response was calculated.

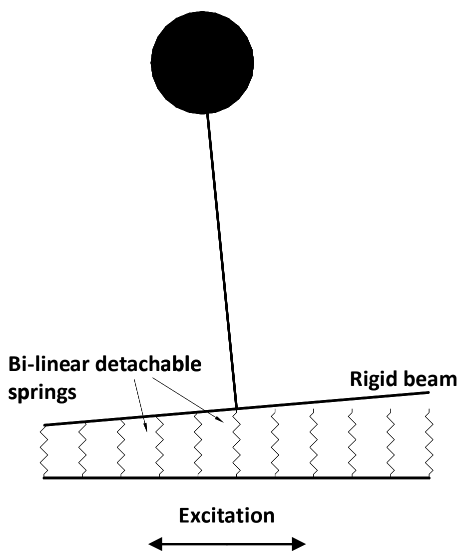

Figure 2. Ruaumoko spring-bed model.

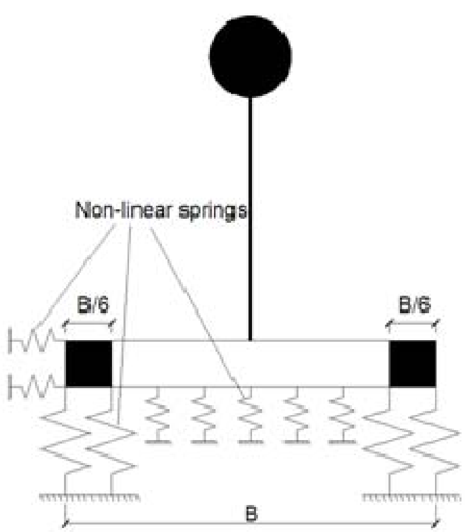

Figure 3. OpenSees spring-bed model with concentration of stiffness towards the outer edges (after FEMA 356).

Figure 5 presents the calculated response of the centrifuge model structure for the three macro-element models. All responses are given at prototype scale. Figures 4 and 5 are presented on one page to aid comparison of the responses. The three columns of figures on the page each have the same plot, so like outputs are found in each column. In Figure 5 the upper row is for the bearing strength surface based macro-element model; the middle row in the diagram is for the Ruaumoko spring bed model; the bottom row is for the OpenSees spring-bed model.

It is apparent in Figure 5 that all three models give a reasonable representation of the observed moment-rotation and horizontal shear-horizontal deformation behaviour. Table 1 summarises the residual displacement at the completion of the centrifuge testing and calculations.

<u>Discussion</u>. The first model used the bearing strength surface for cohesive soils given in Eurocode 8. As the vertical load on the foundation was constant the actions are constrained to a vertical section through the bearing strength surface prior to yielding. This model produces rather "boxy" graphs for the moment - rotation curve and the horizontal shear – horizontal displacement plots. The reason for this is that all behaviour within the yield locus, that is the bearing strength surface, is elastic and nonlinear behaviour occurs only when the action path reaches the bearing strength surface. It would be possible to make the stiffness within the bearing strength surface degrade with the number of cycles, or based on position within the surface as is done by Cremer et al (2001). Another effect not accounted for in this model is uplift at the edges of the foundation. This is the reason for the difference between the shape of the settlement – rotation plot for this model and the other two models. Uplift could be incorporated, see for example by Cremer et al (2001), but that is an additional complication.

The second approach used the detachable, that is no-tension, spring element provided in Ruaumoko. We set the maximum vertical stress on any spring (that is the load carried by the spring divided by the tributary area) to $5.14s_u$; when this pressure is reached the spring yields and from then on the vertical stiffness is reduced. When the direction of loading reverses then the stiffness of the spring reverts to the original value. From Figure 5 it is apparent that this model represents what was observed in the centrifuge reasonably well. The moment rotation curve is plotted without the damping contribution. When this is included the moment rotation curve 'thickens" in the middle and the results is very similar to that for the OpenSees spring bed model.

The final point to make with regard to the Ruaumoko model relates to the rotational stiffness of the bed of springs. We calculated the elastic stiffness of our footing using the Gazetas relations. One can then determine the vertical stiffness of the bed of springs so that it is the same as that for an elastic half space. However, if this is done, then the rotational stiffness from the bed of springs is considerably less than that of the

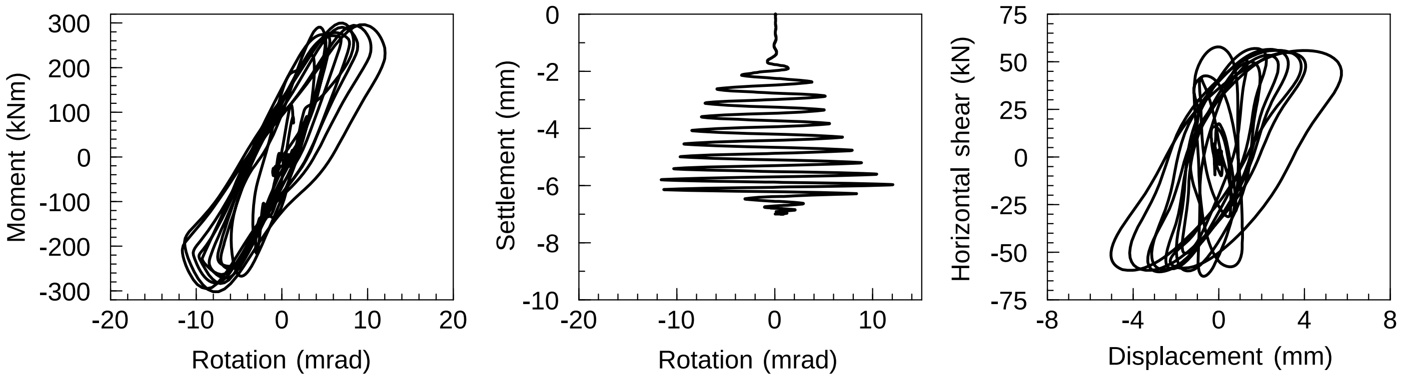

Figure 4 Response measured, at prototype scale, of the model tested in the UC Davis centrifuge.

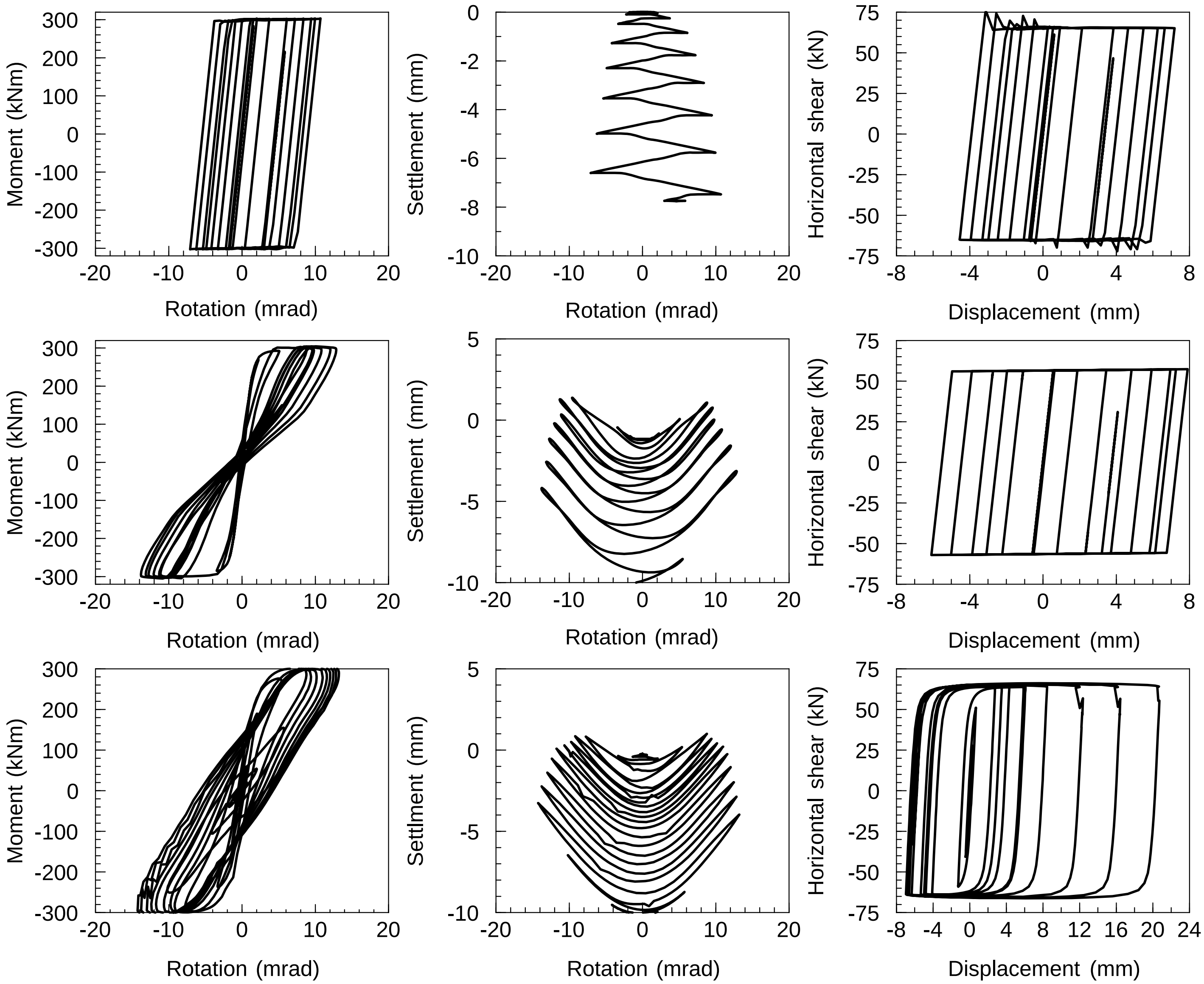

Figure 5 Prototype scale computed dynamic response of the three models to the centrifuge input motion: upper row – bearing strength surface macro-element, middle row – Ruaumoko spring-bed, bottom row – OpenSees spring-bed model.

Table 1: Comparison of prototype scale residual displacements after the centrifuge dynamic excitation.

	Centrifuge measurements	Ruaumoko	Macro element	OpenSees
Settlement (mm)	7.1	10.2	7.7	10.1
Rotation (mrad)	0.8	0.6	4.6	0.03
Horizontal displacement (mm)	0.1	3.8	3.3	5.8

half space. By adding an additional rotational spring to the centre of the footing this deficiency is circumvated.

In the OpenSees model, the calculated moment-rotation plot is quite similar to the measured results. The computed settlement appears to be similar to that of the Ruaumoko model. On the other hand the horizontal sliding range of 27 mm overpredicts the experimental sliding range of 11 mm.

Herein we have concentrated on the residual displacements at the completion of the dynamic loading. Others interested in this approach, Cremer et al (2001), Ugalde et al (2007) and Anastasopolous (2009), have emphasized that another benefit of allowing brief instances of bearing failure beneath shallow foundations during earthquake loading is that the actions applied to the foundation and structure will be less than those on a stiffer foundation. This in turn will lead to economies in design. However, because the foundation is less stiff using this design approach the amplitude of the structural displacements during the earthquake may be as important as the residual deformations, Toh and Pender (2009).

Example – a frame building on shallow foundations

In this example we look at the modeling of a framed building using Ruaumoko to calculate the earthquake response. The work comes from the Wotherspoon (2007) where a more complete account can be found. The details of the structure are given in Figure 6.

The design of a three-storey framed structure with shallow foundations is considered, with the details of the structure illustrated in Figure 6. As can be seen, the structure is five bays long and three bays wide, each bay is 7.5 m by 9.0 m and the storey heights are 3.65 m with the exception of the first storey which is 4.50 m. The shallow foundations were located in a layer of clay with an undrained shear strength of 100 kPa.

The seismic weight on each floor was equivalent to 8.65 kPa, the roof seismic weight was comprised of a 6.75 kPa distributed load and 1000 kN of plant. The basis of these loads was the imposed load required by current New Zealand structural design actions standard, NZS 1170.1 (Standards New Zealand 2002), and the permanent load resulting from reinforced concrete frames supporting prestressed precast concrete floor slabs with 65 mm of site poured concrete topping. Following NZS 1170.5 (Standards New Zealand (2004)) and NZS 3101 (Standards New Zealand (2006)), structural models were designed such that all members contributed to the seismic resistance of the structure and each frame parallel to earthquake propagation had an identical member configuration. Herein we discuss results obtained for an elastic structure, but we have done calculations for a limited ductility (ductility 3) structural models as well. Each floor was modelled as a lumped mass and a rigid diaphragm which restrained the floor such that all points moved the same distance horizontally. All the footings were connected with tie-beams. These were assumed to act under axial load but provide no moment restraint where connected to the footings. For the Ruaumoko modelling Rayleigh tangential stiffness viscous damping was applied to give 5% damping to the fundamental mode and at least 3% damping to every other mode.

Footings 3.1 m square, with the underside 1 m beneath the ground surface, were adopted for all 24 column foundations. Using the load factors given in NZS 1170.5 (1.2 for permanent load and 1.5 for imposed load), bearing capacity calculations revealed that these foundations had adequate to generous bearing strength for the applied static vertical loads.

The fixed base period of the structure detailed in Figure 6 is close to 0.9 seconds.

The specially developed foundation element used in the Ruaumoko calculations has vertical, horizontal and rotational stiffness, all of which are coupled so at uplift all three are detached from the underlying soil. In addition all three springs can exhibit non-linear behaviour. Initially the stiffness of the springs was elastic and gave the settlement under gravity load. To estimate the foundation stiffnesses formulae for the vertical, horizontal and rotational stiffness of rigid rectangular foundations on an elastic soil from Gazetas (1991) were used.

A single earthquake record was used in the analysis and was applied parallel to the longest plan dimension of the structure. This record was from the La Union event, N85W Michoacan, Mexico 1985. The earthquake spectrum was scaled, using the method in NZS 1170.5, to a spectrum representing an earthquake in the Wellington region of New Zealand for a 1 in 500 year return period event. The resulting earthquake time

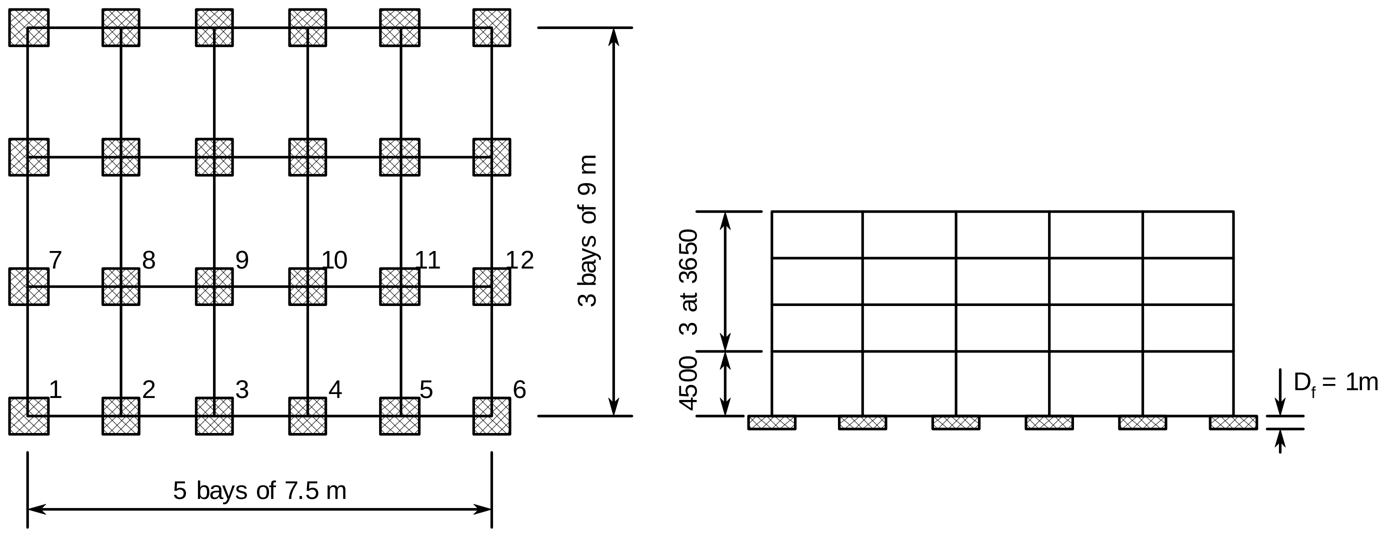

Figure 6 Three-storey structure elevation: plan, and footing numbering.

history had a peak ground acceleration of 3.46 m/sec^2. The response spectrum with 5% damping gives a spectral acceleration at the natural period of the structure of 5.6 m/sec^2.

Initially three methods were used to size the shallow foundations: (i) all footings with adequate bearing strength from static LFRD ultimate limit state considerations, (ii) all footings to have equal static settlement, and (iii) all footings to have equal vertical stiffness with the most heavily loaded footings having adequate static LRFD bearing strength. However, as the bearing capacity of shallow footings decreases rapidly with the application of moment this was found to be the critical design consideration. Whether the structure remains elastic or is designed as ductile, moments are generated at the base of the ground floor columns, and these moments are transferred to the foundation. It was found that only the equal stiffness footings were of sufficient size to accommodate these moments. This appears to give the exterior footings sizes which are extravagant, but although these footings carry the smallest gravity loads they have the largest cyclic vertical loads during the earthquake as well as cyclic shear and moment. Results below were taken from the equal stiffness footing design.

Initial work on this model has been discussed by Pender (2007). There the attention was focused on modeling the uplift of the corner foundations and the effect of moment being applied to the shallow foundations. Further discussion of these effects is given by Wotherspoon and Pender (2010) (this conference). As moment makes substantial demands on shallow foundations it was pointed out that more economical results are obtained if the connection between the column and foundation is pinned. The results presented here extend this consideration to the whole structure, Figs.7 and 8 have maximum moment envelopes over the full height of the corner centre columns. It is immediately apparent that the zero mom-

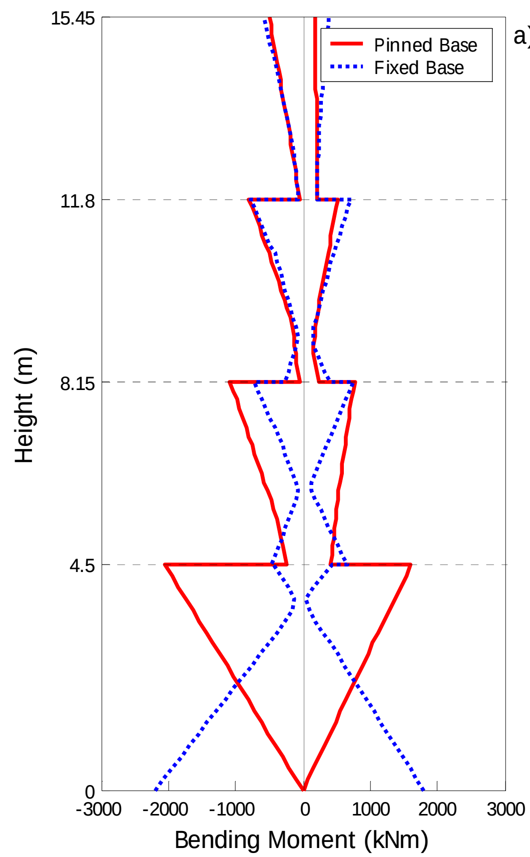

Figure 7 Envelopes of maximum bending moment in columns 1 and 6 calculated for the El Centro 1940 NS earthquake for pinned and fixed column foundation connections.

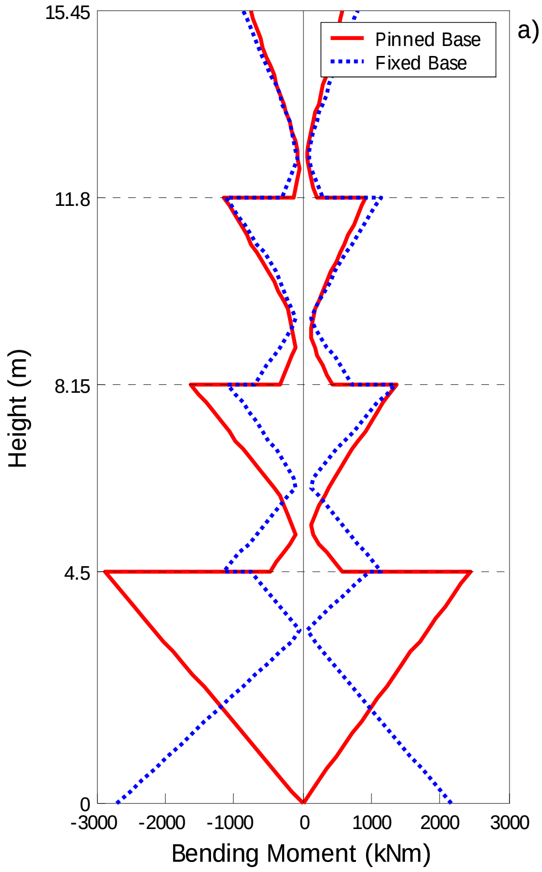

Figure 8 Envelopes of maximum bending moment in columns 9 and 10 calculated for the El Centro 1940 NS earthquake for pinned and fixed column foundation connections.

ent applied to the shallow foundation, when the column-footing connection is pinned, simply pushes the maximum moment elsewhere. These effects continue to the top of the second floor columns. When the column-footing connections are fixed then the largest moment in the column occurs at the footing, when the connection is pinned the largest moment in the column occurs just below the first floor. The magnitudes of these maximum moments are very similar. Thus although the pinned column-footing connection may be more attractive from the point of view of moment demand on the shallow foundations, the maximum moment in the column is hardly changed but the location is altered.

Another important factor is the effect of the column base fixity on the lateral displacement envelope over the building height. This is considered in Pender (2007) where it is shown that the effect is not particularly significant, particularly above the first floor level.

These two shallow foundation examples are illustrations of the interesting insights obtained by considering the earthquake response an integrated model of the foundation structure system.

DEEP FOUNDATIONS

The design of earthquake resistant pile foundations is considered under two headings: kinematic interaction and inertial interaction. The kinematic interaction is related to the deformation of the pile shaft as the earthquake waves propagate upwards through the soil profile. Particularly important are sudden changes in soil properties, especially stiffness, perhaps at layer boundaries, as this is where damage tends to be concentrated. Second there is inertial interaction. This is the excitation applied to the pile head because of the inertial response of the structure supported. In this paper we will discuss only inertial interaction.

A starting point, as in so much geotechnical engineering, is to assume that the soil is an elastic continuum. This has proved to be a fruitful line of approach for soil layers which can be idealized as having uniform properties as there is an extensive range of solutions available for the dynamic stiffness and damping of pile foundations, a summary of which can be found in Gazetas (1991) and Pender (1993).

When the soil profile is less than uniform a common approach is to consider that the interaction between the pile shaft and the surrounding soil can be represented by springs. It has long been established that this is a surprisingly accurate model when the pile is flexible. The decision as to whether this is so is determined by the relative stiffness properties of the pile and soil, refer again to Gazetas (1991) or Pender (1993) which are two examples of many references dealing with this. Most importantly, layering and non-uniformity in the soil can be handled easily using the spring model for pile-soil interaction. Many software packages are capable of pile-spring modeling for flexible piles embedded in layered soil profiles. Herein the use of the Ruaumoko software will be illustrated as this is able to handle nonlinear soil pile interaction, the opening and closing of gaps between the pile shaft and the soil during cyclic loading, and dynamic effects. Figure 9 illustrates the details of the spring model that achieves this, damping is achieved by the hysteresis caused when yielding of the spring

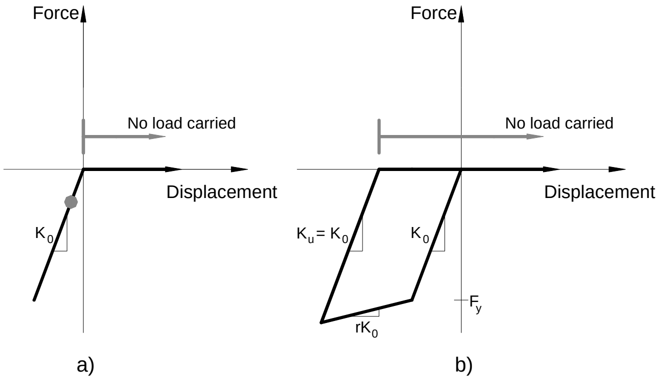

Figure 9 Ruaumoko compression only bi-linear soil hysteresis model indicating displacement ranges where springs carry no force: a) prior to compressive yield, b) after compressive yield.

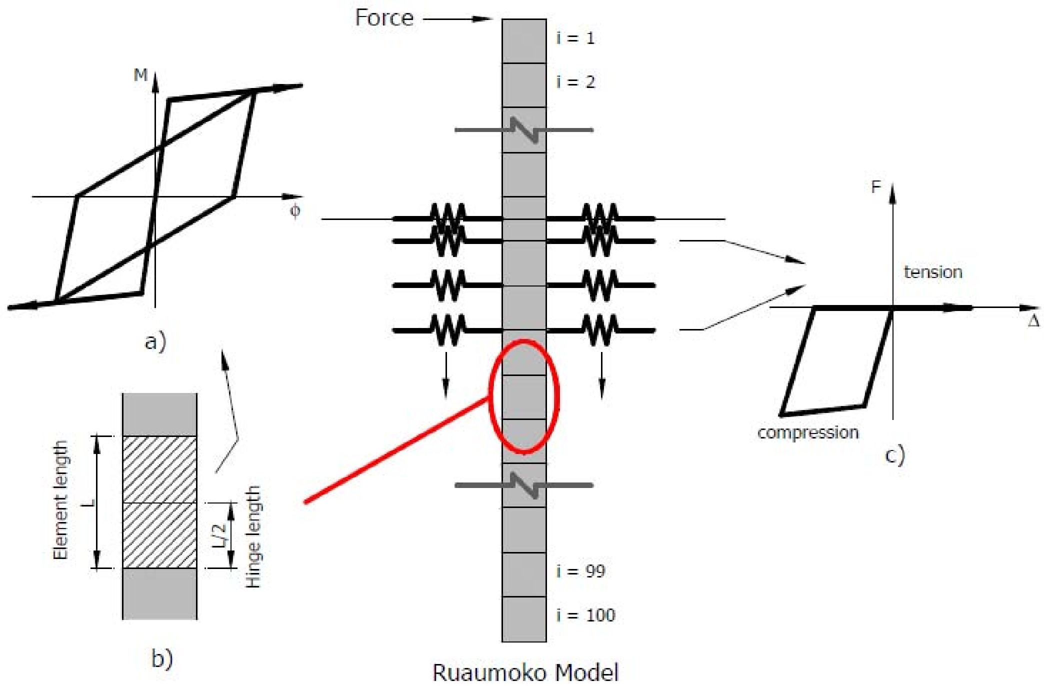

Figure 10 Ruaumoko modeling of pile-soil interaction with non-linear detachable-reattachable springs and yielding of the reinforced concrete pile section.

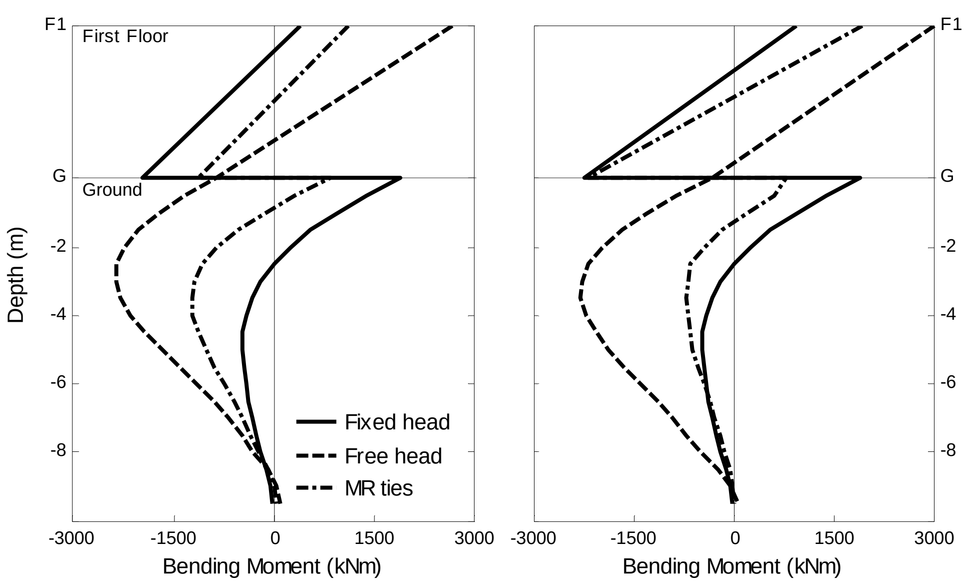

Figure 11 Effect of pile fixity conditions on the bending moment envelopes for the piles and first floor columns of the 10 storey pile supported building. Left: columns 1 and 6, right: columns 9 and 10.

occurs and/or by the addition of dashpot dampers. Furthermore, Ruaumoko is capable of including yielding of the pile shaft. The complete model for the pile-soil interaction is shown in Figure 10.

Recently Ruaumoko, using the model shown in Figure 10, has been applied to analyzing test data from drilled shafts at a site in Iowa where testing took place in the winter with the upper layers of soil frozen and on the same piles in summer when the soil was soft and saturated. Wotherspoon et al (2010a and 2010b) have demonstrated how the Ruaumoko model in Figure 10 is capable of handling these extremes with some success.

When discussing shallow foundations it was stated that liquefiable soil profiles called for pile foundations. The design of piles in liquefiable soil profiles, particularly where the liquefied soil may flow laterally past the pile has become a specialized subfield. Cubrinovski and Ishihara (2004) and Cubrinovski et al (2006) have developed a method of handling this using a pile-spring model having a very small spring stiffness.

Example – framed structure supported on pile foundations.

The structure and foundation system analysed in this part of the paper was a ten storey commercial building supported by single end bearing pile foundations beneath each structural column, the floor plan of the building is the same as that for the three storey building shown in Figure 6; more complete details can be found in Wotherspoon (2007). The integrated system was assumed to be founded on a 25 m thick homogenous clay deposit overlying bedrock. The soil was assumed to remain undrained. For all analyses the undrained shear strength was assumed to be 100 kPa, characteristic of a stiff clay deposit. Young's modulus of the soil was assumed to be 50 MPa and the unit weight of the soil was 18 kN/m^3.

The ten storey building was designed as a moment resisting reinforced concrete frame using current New Zealand design standards (NZS 3101, SANZ (2006)). It was assumed to be a commercial building in which all members contributed to the seismic resistance of the structure, with each frame designed with identical member sizing. The reinforced concrete frame supported precast concrete floor slabs with 65 mm of site poured concrete topping. The roof of the structure was also constructed of reinforced concrete, adding an additional level of seismic mass to the structure. The ground floor concrete slab was poured to a depth of 125 mm over the entire structural footprint, and was ignored in the analysis as it was assumed to provide minimal additional stiffness.

This structure was designed as nominally ductile with a ductility of 1.25 in order for the structure to remain elastic under seismic loading. To account for the effect of cracking on member stiffness, the effective moments of inertia (I_{eff}) of the member sections were calculated using modifications to the gross moment of inertia (I_g) defined in NZS 3101:2006. Beams and columns had dimensions of 900 x 500 mm and 850 mm square respectively, with a Young's modulus of 26500 MPa corresponding to concrete with 35 MPa unconfined shear strength.

The foundation system consisted of CIDH (Cast in drill hole) end bearing piles with a depth to bedrock of 25 m. Designs were developed for single piles beneath each of the columns, which were intended to remain elastic during loading. For simplicity, all piles beneath the structure had a 1000 mm diameter pile section and were constructed of 40 MPa concrete. Apart from the connection created by the tie-beams, it was assumed that each pile had no significant influence on any of the other piles. This assumption holds as long as the failure zone for each pile does not overlap, controlled by the minimum centre to centre spacing between adjacent columns of 7.5 m.

The north-south component of the El Centro (1940) record, scaled, using the methodology of NZS 1170.5 (Standards New Zealand 2004), for a 500 year return period event in the Wellington region at the fundamental period of the structure-foundation system was applied at all point along the pile shaft.

The calculated maximum moment envelopes in the upper parts of the piles and in the first floor columns are shown in Figure 11. These profiles are shown for three conditions: fully fixed column-pile connections, pinned column-pile connections and moment resisting tie beams connecting the tops of the piles. The conclusion derived from this figure is not dissimilar to that in Figs. 7 and 8, namely that the fixity between the bottom of the columns and underlying foundation has a very significant effect on the moment distribution in the columns for the first few storeys. This provides the designer with options that become clear when an integrated model of the structure-foundation system is analysed.

SITE INVESTIGATION AND GEOTECHNICAL CHARACTERISATION OF SOIL PRESENT

Above the discussion and examples have been about the need for developing a comprehensive numerical model of the foundation-structure system with the intention of achieving a more economical design. However, it must not be overlooked that a very important part of the process is obtaining the best quality information possible about the soil profile in which the foundation will be constructed. The increase in sophistication implied in the integrated approach to structure-foundation design can only successful if it is matched by equal sophistication in the understanding of the soil profile and the properties of the materials present.

Quite properly current site investigation techniques make extensive use of penetration testing, both the Cone Penetration Test and the Standard Penetration test. Of these the CPT provides more information than the SPT, but in coarse grained and gravelly deposits the SPT, or even something heavier, is required. An important feature of the CPT is that it provides more or less continuous data about the soil penetration resistance and other properties, thus changes in properties, layering and variability within a given soil layer or geological formation are clearly displayed. In turn this is an important insight into the soil properties. In addition to these devices which give "point" values for some soil property, or properties, geophysical techniques have recently come of age as viable methods for measuring representative values for shear wave velocities of soil volumes affected by the foundation, Stokoe et al (2004).

Geophysical site investigation techniques can give estimates of the small strain shear modulus of the soil layers. This is an important parameter because it provides an upper bound on the soil stiffness. Early proponents of these methods emphasized that the soil stiffness obtained from geophysical investigation, that this the small strain stiffness, was an appropriate value for estimation of the settlement of shallow foundations under static loads. However, the small strain shear modulus is not actually what is required for earthquake resistant foundation design as strains in addition to those induced by the static loads will occur beneath the foundation. This will lead to softening of the soil and the effective stiffness will be less than the small strain value. Thus it is necessary to account for the strain dependent nature of soil stress-strain behaviour. One possibility is to perform a three dimensional finite element analysis of the foundation, the surrounding soil, and the structure attached to the foundation in which the nonlinear stress-strain behaviour of the soil is incorporated explicitly. Examples of this approach are Kramer (2009) and Jeremic et al (2009). Useful as these approaches are, they are hardly design tools and so the examples provided herein have been based on the use of simpler models. An appealing alternative is suggested by EC8 Part V (Table 4.1) in which the equivalent soil stiffness is a reducing fraction of the small strain shear modulus as the peak ground acceleration rises. This approach envisages a type of equivalent linear representation of the soil stiffness which decreases as the ground acceleration increases whilst the amount of damping increases.

Field tests to evaluate the effective soil modulus

Field testing with one objective being to investigate the extent of the softening of the ground stiffness under cyclic loading of foundations on residual soil in Auckland has been reported by Sa'don et al (2009) for pile foundations and by Algie et al (2009) for shallow foundations. Some results from this testing are summarized below. These foundations were subject to sinusoidal excitation from an eccentric mass shaking machine. In that the foundations were excited from above, the insights gained are relevant to the inertial interaction part of the soil-foundation-structure interaction.

The initial stages of this work involved a thorough site investigation. Intense cone penetration profiling of the site was done and supplemented with seismic CPT work. In addition WAK testing (Briaud and Lepert (1990)) was done to establish the soil modulus distribution near the ground surface. The values for the small strain shear modulus determined by these methods were consistent and furthermore they were consistent with the rotational stiffness of the shallow foundations at low level of excitation and the lateral stiffness of the pile heads at low levels of excitation. Laboratory testing on samples of similar soil confirmed the values for the small strain shear modulus and provided information about the degradation of stiffness with increasing strain. So from this initial investigation we are confident that the stiffness properties of the residual soil at the site have been measured accurately.

Shallow foundation tests. A framed structure was constructed that could be mounted on shallow reinforced concrete foundations 2 m long and 0.4 m wide, the underside of which was 0.4 m beneath the ground surface. At the site 8 of these foundations were constructed. A demountable structure fabricated from structural members was made that could be erected on pairs of the foundations. Figure 12 shows the structure with additional mass and the shaking machine attached. The height of the end frames was 3 m and the span of the top frame was 6 m. The total weight of the structure, shaking machine and loading plates was about 200 kN. The structure was instrumented with strain gauges so that the loads applied to the foundations could be monitored, accelerometers, displacement transducers and pressure cells beneath the foundation.

Figure 12 Shallow foundation test structure with shaker in place and additional weight added.

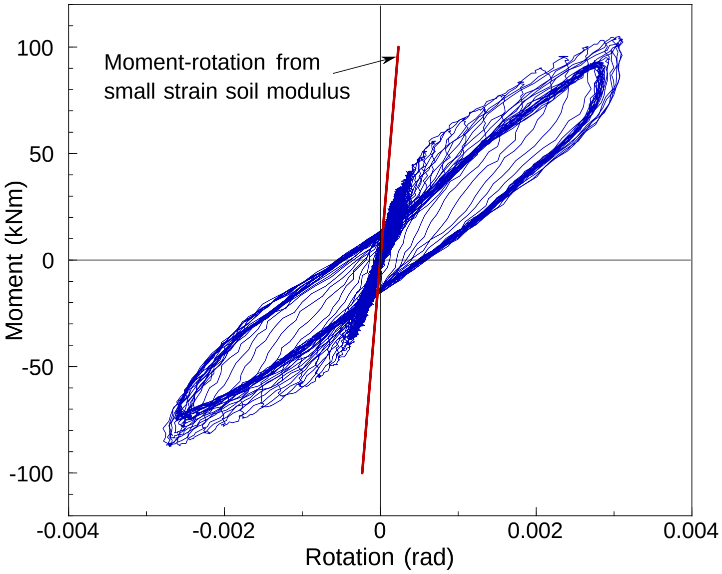

Figure 13 Measured moment-rotation loops at excitation frequencies between 2.0 and 2.5 Hz. (Initial static bearing strength factor of safety for the shallow foundations: 10)

For the tests discussed here the vertical load on the foundations was such that the static bearing strength factor of safety was about 10. The testing sequence followed was first to subject the structure to low intensity excitation to obtain the small strain natural period of the system. This was followed with high level excitation, and finally the low intensity excitation was repeated to check on the change to the system caused by the high intensity shaking.

The results from part of the shaking are shown in Figure 13; this covers several seconds of shaking as the frequency of excitation was moved from 2.0 to 2.5 Hertz and a new steady state response established. Clearly the number of cycles applied at each frequency is well in excess of the number expected during a design earthquake. The calculated moment capacity of the footings with the applied vertical load is about 110 kNm, thus at the 2.5 Hertz frequency level the footings are close to mobilizing all the bearing strength. It is noticeable that there is a big change in rotational stiffness of the footings going from 2.0 hertz to 2.5 hertz. This is not a frequency effect but simply a consequence of the increase in the sinusoidal force at the higher frequency. Also marked in the diagram is the rotational stiffness of the footings calculated using the small strain stiffness of the soil. Clearly the design of footings to resist earthquake actions needs to use an apparent stiffness for the soil that is much less than the small strain value. During these tests the vertical settlement and horizontal displacement of the foundations were monitored. In both cases these movements were very small, so the main response of the system to the excitation was in rocking.

<u>Cyclic pile tests</u>. Four steel pipe piles were driven, closed-ended, to a depth of about 7.0 m into the soil classified as residual clay. The piles have an outside diameter of 273 mm and a wall thickness of 9.3 mm with pile lengths of 7.5 m. Piles 1 and 4 were instrumented with ten pairs of waterproof strain gauges along the length of the pile up to 7 m depth in order to measure flexural strains and moments during loading (these gauges were protected by tack-welding pieces of steel angle to the piles). Two of the strain gauge pairs were located above the ground surface at 0.4 and 0.6 m to estimate the applied actions applied by the shaker. A pile with shaker attached is shown in Figure 14.

The forced-vibration tests were conducted just after the winter wet season, so that the soil can be assumed to be saturated to the ground surface. Also, before the tests started, the top soil surrounding the pile was carefully removed by using hand spade up to 150 mm depth to provide good clearance for observing pile-soil gap opening. The distance from the top of the pile to the ground surface adjacent to the pile was 1 m.

Before starting the shaker the response of the piles was measured with a gentle blow with an instrumented hammer. The natural frequency so obtained was close to the value calculated with the small strain shear modulus of the soil. Next the shaker was taken through the frequency range with very small mass. This produced the response curve, Figure 15, that had a natural frequency of 11 Hertz, close to that obtained using the small strain stiffness of the soil. This was followed with a higher intensity of shaking, the moment-rotation curves for which are shown in Figure 16. After the high level shaking the natural frequency was again determined using low level shaking and the frequency had decreased to 8.2 Hertz. After the test gaps were apparent around the pile shaft at the ground surface, the depth of these was more than one pile diameter as observed by pushing a piece of metal measuring tape down into the gap.

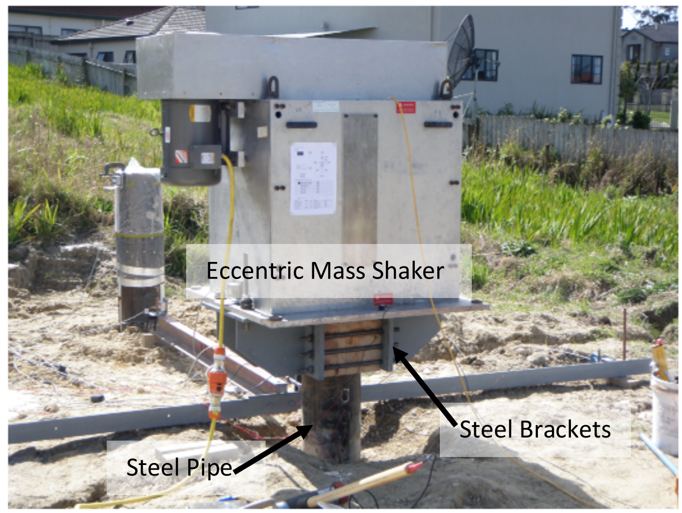

Figure 14 Pile with shaking machine mounted.

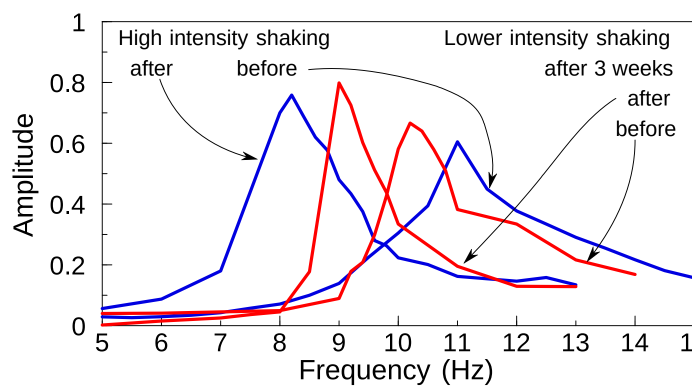

Figure 15 Pile head response curves before and after shaking.

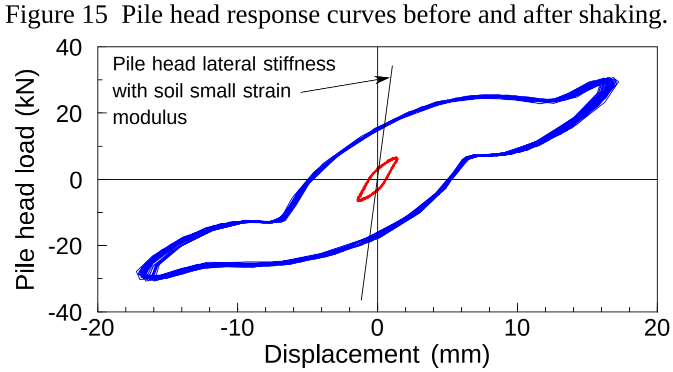

Figure 16 Pile head load-displacement loops for gentle and higher intensity shaking.

After resting for three weeks, during which time there was some rain, the gaps adjacent to the pile shaft had disappeared.

The test sequence was repeated and the natural frequency was now 10.2 Hertz, this was reduced to 9.2 Hertz after some mild intensity of excitation.

The moment rotation curve for the first high level shaking is shown in Figure 16. Also marked on this figure is the rotational stiffness of the pile head calculated using the small strain shear modulus of the soil. As with the shallow foundation the cyclic stiffness of the pile head is much reduced. Even when the cyclic force is only +- 7 kN the apparent stiffness is about 7 times softer than the small strain value. When the cyclic shear is increased to 31 kN there is a further reduction is stiffness and also the effect of the gapping adjacent to the pile shaft becomes evident. The bumps part way along the loop are thought to be caused by the pile shaft engaging the other side of the gap.

These field tests are not completely relevant to earthquake loading because of the large number of cycles. However, the important conclusion from both test series is that the appropriate soil stiffness for earthquake resistant foundation design is much less than the small strain value. So and important part of the design process will be choosing an "operational" modulus value. This conclusion thus supports the thinking behind the provisions of Table 4.1 in Part V of Eurocode 8.

Note that the apparent change in the ground stiffness revealed in the above two sets of tests is caused by the change in strain levels in the soil, it has nothing to do with variability of the soil at the site. The FEMA 356 document (FEMA 2000) addresses site investigation very briefly. It calls for geotechnical reports to state clearly the basis on which values for soil parameters such as shear strength and stiffness are derived. This suggests an underlying assumption that the site investigation work is done quite separately to foundation design and even that the personnel responsible for the investigation are unlikely to be involved in the foundation design. Clearly, this is not a process that promotes effective interaction between the geotechnical and structural communities. Another interesting feature of the FEMA document is advice to perform three sets of calculations. The first with the best estimate of the soil property values, the second with these values halved and a third with these values doubled. The reasoning behind this recommendation is not explained in the document. Presumably the halving of the value is to check on the effects of the ground being softer than expected. In foundation subject only to gravity loads one would hardly bother about doubling the estimated ground properties. A possible explanation for this in an earthquake resistant context might be that underestimation of ground properties could, depending on the details of the earthquake, lead to an underestimation of the true foundation actions. Halving and doubling suggests that site investigation techniques are a very long way from being able to provide reliable information. Halving and doubling the mean values implies that the coefficient of variation for the soil properties is 1.0. This is an exceedingly pessimistic assessment as the limit of the range is typically about 0.5 (Baecher and Christian, (2003)). However, this suggestion could be viewed as a way of deciding if further investigation is necessary – if the system performance is not sensitive to these extreme variations, then all is well.

CONCLUSIONS

The main purpose of this paper is to summarise the current state of an ongoing project promoting the integrated design of structure foundation systems. By this is meant that the earthquake resistant design requires the analysis of a **system** that has the foundation and structure modelled with comparable levels of sophistication. To achieve this requires close collaboration between the structural and geotechnical teams involved on a project.

The material covered extends beyond traditional soil structure interaction (SSI) which is usually restricted to consideration of elastic soil behaviour. The intention of modelling the foundation and structure as a single entity is that nonlinear behaviour of the soil supporting the foundation leads to design economies as the earthquake actions applied to the structure are reduced. The term given to this approach is Soil-Foundation-Structure-Interaction (SFSI).

Although it is feasible to perform a detailed finite element time history analysis of foundation behaviour including nonlinear soil and structure response, the emphasise in the paper is on simpler models that have the potential to be useful in a design office.

However, to achieve a truly integrated structure-foundation design the investigation of the soil in which the foundation will be constructed needs to receive effort comparable to that expended in modelling the structure-foundation system.

Several examples are given in the paper:

- First, comparisons between results from dynamic tests on a shallow foundation in a centrifuge and numerical modelling of the response. The conclusion from this was that the calculated permanent deformations of the foundation models reasonably well the centrifuge results. Furthermore the end result seems not to be sensitive to the way the soil beneath the foundation is modelled.

- Second, it was shown that for a three storey framed building on shallow foundations, the moment distribution in the first and second floor columns was very dependent on the fixity of the connection between the bottom of the column and the footing.

- Third, for a ten storey framed building on pile foundations it was found that the relative stiffness of the piles, ground floor columns and tie beams determines the moment distribution in the columns of the bottom two floors.

The final part of the paper presents results of field testing of pile foundations and shallow foundation. These show that the operational stiffness of the foundations during cyclic loading is a great deal less that the stiffness that would be expected if the small strain stiffness of the soil was the controlling parameter.

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