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COMBINED MODELLING OF STRUCTURAL AND FOUNDATION SYSTEMS

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SUMMARY

The work reported in this paper illustrates our conviction that the route to more effective design of structure/foundation systems is through better cooperation between structural and geotechnical specialists. To this end we demonstrate the use of an existing dynamic structural analysis package in modelling the earthquake response of framed structures on shallow foundations. The work presented in this paper is a continuation of that in Wotherspoon et al. [1]. The buildings analysed are multistorey reinforced concrete framed structures founded on discrete shallow footings, connected with tie beams (not part of the foundation). Yielding and uplift characteristics of the foundations have been modelled by adapting available structural models in the software. The underlying soil is stiff clay.

Three methods were used to size the shallow foundations: all footings having a static bearing capacity factor of safety of 3.0, all footings having equal static settlement, and all footings having equal stiffness with the most heavily loaded footings to have a static bearing capacity factor of safety of 3.0. However, the fact that the bearing capacity of shallow footings decreases rapidly with the application of moment was found to be the critical design consideration. Considerable moments are generated at the base of the ground floor columns for both structures designed to remain elastic and for those exhibiting ductile behaviour. These moments are transferred to the foundation and it was found that only the equal stiffness footings were of sufficient size to accommodate these moments. An additional analysis to emphasise the effect of moment loading on the shallow footings was undertaken with the structures modelled with pinned connections at the base of the columns. This meant that no moment was transferred to the shallow foundations and the demands on the footings from the earthquake loading were found to be modest.

INTRODUCTION

A problem endemic in design of the built environment is poor communication between structural and geotechnical specialists. This is a consequence of ever-increasing fragmentation of the engineering profession into sub-specialisations. The structural designer has a sophisticated understanding of construction materials, whereas the geotechnical engineer is expert in the properties of the soil and rock

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masses on which structures are founded. The absence of a team approach is a potential source of confusion and/or inefficiency in structure/foundation design.

Furthermore, changing demands placed upon designers by owners and users of the built environment dictate that procedures continue to evolve to match social expectations. Recent devastating natural disasters and terrorist attacks have led to demand for superior performance of both existing and new infrastructure. Assessing accurately the existing state of foundation systems is particularly demanding. By considering the structure and foundation as an integrated system, new opportunities may arise for achieving superior performance, and the demonstration of this is the purpose of this paper which extends the discussion of Wotherspoon et al. It is envisaged that integrated design of structure/foundation systems will lead to improvements in an environment where performance based criteria are the norm, and also in making more accurate assessment of the available capacity of existing structures and foundations when retrofit is being considered.

Analyses are presented for a suite of moment resisting reinforced concrete frame structures, designed to resist earthquake loading, and supported on shallow foundations resting on a stiff clay. Computer modelling was undertaken using RUAUMOKO [2], a nonlinear dynamic structural analysis program. Various foundation and structural characteristics were investigated to demonstrate effects on the behaviour of the whole system. Structures on shallow foundations were considered and the effects of structural yielding, nonlinear soil behaviour and foundation uplift were determined.

The question we are addressing is what is the best approach for the design of shallow foundations of multistorey framed structures. One approach is to follow a loading standard. Documents in use in New Zealand imply that the maximum bearing capacity demand should not exceed more than about half the available ultimate capacity. However, this considers each foundation separately and does not give a view of the behaviour of the total structure/foundation system. Our intention in using RUAMOKO to examine the performance of the whole system during earthquake time histories was to overcome this limitation. To date our approach has been to make use of different structural member elements in RUAMOKO and investigate how they can be adapted to model foundation behaviour. In our earlier paper we illustrated how various structural elements, particularly yielding springs, can represent aspects of foundation behaviour. Herein we explore this further and illustrate some of the limitations of this approach, as there is a complex interaction between the applied vertical load, horizontal shear, moment and the bearing capacity of a shallow foundation. We have found it possible to model uplift of the separate foundations using RUAMOKO. This means that the footing carries no vertical load, but the model does not allow us to detach the shear and moment springs at the instant of uplift, so that at this time accurate moment and shear behaviour is not developed. We illustrate below the extent of this infringement of accuracy.

BACKGROUND

Soil structure interaction has long been an important topic in the design of structures to resist earthquake loading. Generally this has considered elastic soil behaviour, focusing on the effect of loading frequency and the characterisation of damping. Simple models have been developed by Wolf [3]. In addition, Stewart et al. [4, 5] and Trifunic [6] have reviewed the effectiveness of linear elastic soil structure interaction modelling, finding from recorded building response that these methods are a suitable design tool, at least for mild earthquakes. Alternative approaches, focusing more on design calculations, were presented by Pecker and Pender [7] and Martin and Lam [8].

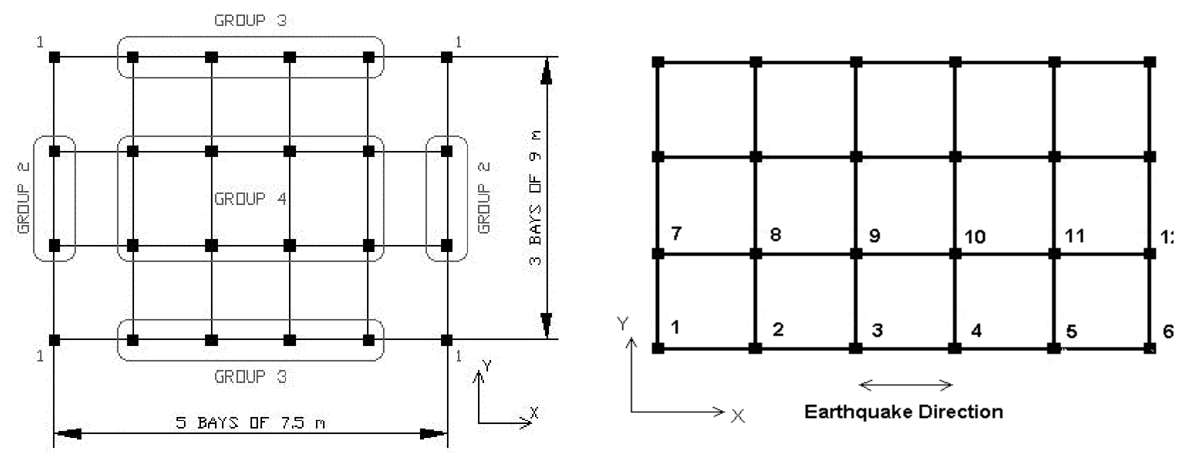
To achieve a satisfactory shallow foundation design the ultimate capacity of the foundation needs to be considered, as well as the stiffness and the decrease in stiffness as the ultimate capacity is approached. The ultimate capacity of the foundation is a complex function of the vertical load, horizontal shear and moment. For a given vertical load the presence of shear and moment reduce the capacity. As it is not possible to generate tensile stresses between the underside of the foundation and the soil, moment is resisted by a couple generated by offsetting the location of the vertical reaction on the underside of the foundation. This reduces the effective area of the foundation, and as a consequence the ultimate capacity of the foundation is very sensitive to moment.

In addition to the combined action of vertical load, shear, and moment, the bearing capacity is also reduced by inertia effects in the soil beneath the foundation. As this effect is small for foundations on clay it is not considered in this paper.

A bearing capacity factor of safety of 3.0 is commonly used for the design of shallow foundations under static load, while for short term earthquake actions the bearing capacity factor of safety is allowed to fall to about 2.0. The equivalent result is achieved in load and resistance factor design (LRFD) by applying load factors of about 1.5 for static loading and 1.0 under earthquake loading, whilst for both these conditions a strength reduction factor of about 0.5 is applied to the ultimate bearing capacity.

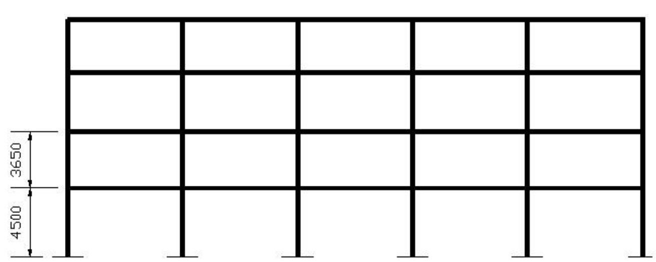
STRUCTURE MODELS

Previous research was completed by the Building Research Association of New Zealand (BRANZ) on a suite of structural models for use in development of the AS/NZS 1170 Draft Loading Standard [9], and these models were also used as the basis for the study reported in this paper. Limited ductile displacement (ductility 3) structural models used in this analysis were developed by Compusoft Engineering [10], with seismic analysis and member design of the suite of structures determined using the ETABS [11] finite element analysis program. Structural members were designed in accordance with the AS/NZS Draft Loading Standard, NZS 3101 [12] and NZS 4203 [13].



(a) Footing groups **(b) Footing numbering system**

Fig. 1: Three storey structure elevation and floor plan indicating footing groups and numbers



(c) Structure elevation

Models were designed such that all members contributed to the seismic resistance of the structure and each frame parallel to earthquake propagation had identical member configurations. Models of three dimensional three and ten storey reinforced concrete moment resisting frame structures were analysed with both fixed base and compliant foundations. These models had the generic building plan shown in Fig. 1, with the first floor height being 4.5 m, and the remaining storey heights fixed at 3.65 m. The seismic weights for the three and ten storey buildings were 8.5 kPa loading at each floor, while the roof load was 7.0 kPa plus a 1000 kN for plant. The basis of this was the live load required by the current loading standard NZS 4203 and the dead load resulting from reinforced concrete frames supporting prestressed precast concrete floor slabs with 65 mm site poured concrete topping.

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. The ductile structure was modelled to develop a sway mechanism under seismic loading, which was achieved by permitting column hinging at ground level and beam hinging at locations within 1.5 beam depths from the column faces.

RUAUMOKO models were developed using the same assumptions as above to undertake detailed nonlinear analysis. Columns were modelled using concrete beam-column frame members, and beams were modelled using Giberson beam frame members. The Giberson frame members consisted of an elastic central section with potential plastic spring hinges located at the ends. The concrete beam-column members were of similar form, but instead the central section was defined by a beam column yield surface. As with the ETABS design model, column hinges were restricted to ground level in the RUAUMOKO model. The Modified Takeda Hysteresis model (see Fig. 2) [14] was used to represent the hysteresis of structural members during nonlinear analysis. Hinge lengths were taken as two thirds of the member depth.

FOUNDATION MODEL

The shallow foundations were assumed to be founded on clay with an undrained shear strength of 100 kPa. This value is typical of Auckland residual clay which has vane shear strengths in the range of 70 to 120kPa. For earthquake loading the undrained shear strength will be greater because of the rate of loading. Pender and Ahmed-Zeki [15] observed an increase of about 40% in the undrained shear strength of this soil when tested at rates of loading comparable to those during a seismic event. Thus the value of 100kPa used herein for earthquake loading was equivalent to the lower end of the range of values found in normal site investigation.

The appropriate stiffness to use for this material requires consideration as the load deformation behaviour of a shallow foundation is nonlinear both for static and dynamic loading. At a very small level of vibration the stiffness will be controlled by the shear wave velocity of the material, providing an upper limit to the stiffness. For Auckland residual clays one would expect a shear wave velocity of about 150 to 175 m/sec, which corresponds to a Young's modulus of the soil of about 150 MPa. A commonly used empirical relation is that the Young's modulus of a clay is about $500s_u$, which is about 50 MPa for this material. This modulus is likely to represent the soil stiffness corresponding to settlement of the foundations under long-term static load. As the foundation responds to earthquake loading there will be a further reduction in the apparent modulus of the soil as more of the available shear strength of the soil beneath the foundation is mobilised. Clearly a proper load deformation relationship for a footing would be nonlinear and exhibit hysteresis under dynamic loading. Lacking this we have used a bilinear model with an equivalent secant modulus and modelled the load deformation behaviour of the footing as a spring member with a Young's modulus of 10 MPa. (Of interest is the Young's moduli measured on block specimens of Auckland residual clay, where values in the range of 10 to 50 MPa are found from laboratory testing.)

At the outset of the work it was not clear how most appropriately proportion the footings for the static load on the foundations. We considered three approaches. Firstly, all footings were designed to have a static bearing capacity factor of safety of 3.0. Secondly, the footings were proportioned so that there was constant static settlement of the foundations, and finally the footings were sized to have the same stiffness. Footing sizes for the equal static settlement method were determined by designing the corner footings to a static bearing capacity factor of safety of 3.0. The static settlement that developed for this footing size was then applied to the other footings to determine their required size in order to achieve an even static settlement value across the structure. For the equal stiffness approach the footings beneath the central columns, which carry the largest load, were proportioned to have a static bearing capacity factor of safety of 3.0, and all other footings were given the same dimensions (and therefore stiffness). Footings were connected by elastic tie beams, restraining the footings so that all had the same horizontal movement. The tie beams were assumed to be attached to the footings without rotational restraint so that they would transfer axial forces only.

Initially, for comparative purposes a set of calculations was made with the structural models attached to rigid foundations, such that all degrees of freedom at the column bases were fully fixed. Three methods were used to model soil-structure interaction of the foundations on the soil profile. First the

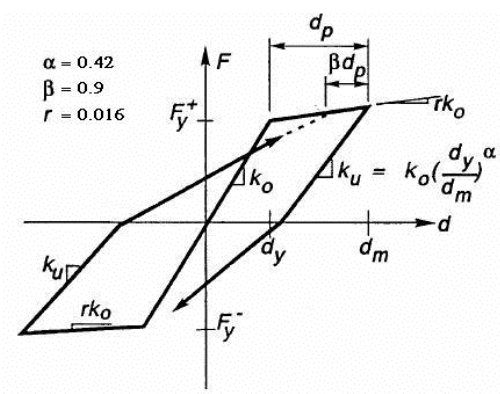


Fig. 2: Modified Takeda Hysteresis Model (after Carr 2002) used for structural yielding.

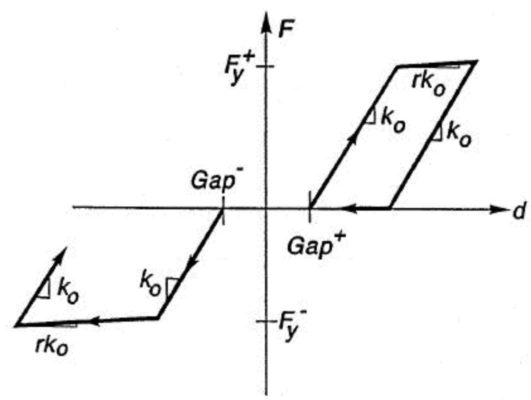


Fig. 3: Bi-linear with Slackness Hysteresis model (after Carr 2002)

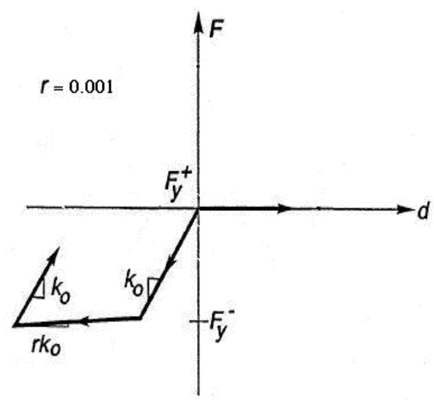


Fig. 4: Modified Hysteresis model describing foundation yielding and uplift (uplift occurs to the right of the origin and elastic-yielding displacement in compression to the left of the origin).

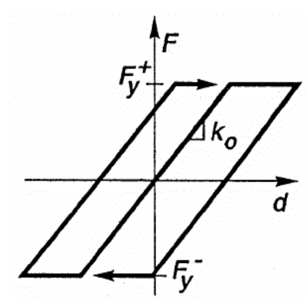


Fig. 5: Elasto-Plastic Hysteresis Model (after Carr 2002) used to describe the horizontal yielding of the shallow foundations.

methods of Gazetas et al. [16, 17, 18] were used to estimate the vertical, horizontal and rocking stiffness of the foundations. These all assumed elastic behaviour of the soil layer and accounted for the effect of embedment and sidewall contact on the stiffness. Interaction between the footings and the ground below was represented in this way by using three independent springs.

For nonlinear foundation response, the soil hysteresis was modelled using structural hysteresis models available in the RUAUMOKO library. For vertical actions the Bi-linear with Slackness Hysteresis model was utilised (See Fig. 3). Input factors for the hysteresis model were applied so that footing uplift could be represented. Both gap lengths were set to zero to develop a continuous hysteresis loop. To represent uplift the positive yield value (F_y^+) of the soil springs was set at zero. Positive forces in the spring were tensile, therefore when the spring force reached zero no tensile load was carried by the spring, while still being free to move in the uplift range. This modified version of the soil hysteresis model in Fig. 3 is shown in Fig. 4. Initial vertical footings stiffness was estimated using formulae derived by Gazetas, Dobry and Tassoulas [16]. In the negative, or settlement direction, the yield force (F_y^-) was set at the value corresponding to a bearing capacity factor of safety of unity.

Horizontal stiffness was calculated using the methods of Gazetas and Tassoulas [17], with dynamic response of the horizontal soil spring represented by a simple elasto-plastic model (Fig. 5). The positive and negative yield values of the footings were developed from a failure mechanism where resistance to yield was developed through base shear and passive resistance at one end of the footing. Rotational stiffness of each footing was calculated using the methods of Hatzikonstantiou et al. [18].

For discrete footings the question of interaction arises for both stiffness and bearing capacity. The equal stiffness footings were the largest, being about 3 m square for the 3-storey structure. From Fig. 1 it is seen that the bay size for the frames is 7.5 m by 9.0 m. These dimensions give clear spaces between the footings of 6 m in one direction and 7.5 m in the other, and these were considered sufficiently large to assume that interaction would be negligible. In the case of the 10-storey structure the equal stiffness footing size was 5m square, so in this case the clear space between the footings was 4 m in one direction and 1.5 m in the other. At these footing spacings some interaction is likely, although this was not considered herein.

Moment and Vertical Force Coupling

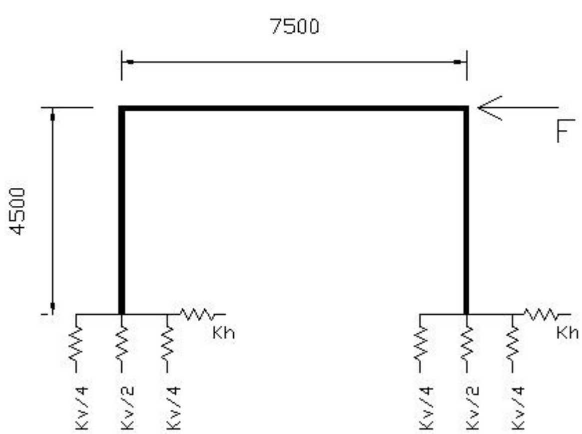
Our first calculations were made using a three independent spring model to represent the shallow foundation, and this uncoupled vertical, horizontal and moment spring model is identified as the Gazetas spring model throughout this paper. The first exercise was to check the bearing capacity of the shallow foundations when each load component was at a maximum, so when the vertical load was a maximum the corresponding shear and moment on the foundation were noted. Similar load combinations were gathered when the shear and moment were at a maximum. Using these combinations the bearing capacity was checked for the 3 m square foundations for the 3-storey structure and 5 m square foundations for the 10-storey structure. We found that the smallest values for the bearing capacity factors of safety were approximately 1.3 for the 3-storey structure and 2.85 for the 10-storey structure.

If footing uplift occurred in the Gazetas spring model the vertical spring would detach. However, because of the independent springs used in this model, the footing still had moment and shear applied. To remedy this problem when an uplifted footing still carried moment and shear we looked at a second model for the shallow foundation. Instead of independent vertical and moment springs we used three vertical springs as shown in Fig. 6. The stiffness and spacing of the springs were designed to give the correct vertical and rotational stiffness of the shallow foundation and the vertical springs were modelled using the same elements as used for the Gazetas vertical spring discussed above. The behaviour of this model was investigated by performing a push-over analysis on a single bay portal frame, resulting in the multi-linear load deformation behaviour in Fig. 6. This relationship is taken from the point of load application.

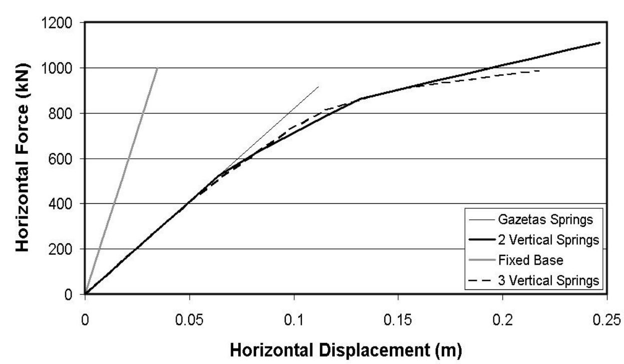
The Gazetas spring model (uncoupled vertical, shear and moment springs) followed a linear load deformation relationship due to the use of only one vertical spring. For this simple structure the relationship could not be bilinear because when uplift of the single spring occurred, the structure detached from the ground and had no restraint. Fig. 6 shows that the models that incorporated two and three vertical springs developed a multi-linear load deformation relationship due to the progressive uplift of the springs as load increased. The three vertical spring model had more changes in stiffness as the extra spring allowed for an additional uplift event to occur. The same multi linear relationship was evident in the comparison between horizontal force and vertical displacement.

This response also indicated the significant influence that the foundation model had on the horizontal stiffness of the structure/foundation system. The inclusion of a foundation model in the simple portal frame analysis decreased the horizontal stiffness to less than half the stiffness of a fixed base structure. The main deficiency in this model was that the shear stiffness of the footing did not change when uplift occurred. We have not yet determined how to form the model using RUAUMOKO such that shear resistance drops to zero when the footing lifts off. However, this is not a major problem at present as the shear developed in the analysis detailed in the results section below can be resisted through passive pressure at the front of the footing. This prevents the base of the footing from having to resist any shear and eliminates any detrimental effect to the bearing capacity of the footing.

The final foundation model was developed by adopting pinned connections between the footings and the base of the columns. This was done because the moment capacity at the base of the columns at zero axial load was approximately 1000kNm for both the 3-storey and 10-storey structure. For both of the above structures the calculations indicated that the peak moments at the base of the columns were at or close to the moment capacity of the columns. In addition, the base moment reduced the bearing capacity of the foundation, but when the foundation was about to lift off results indicated that moment was still applied to the footing. Consequently, a third set of calculations were conducted to establish a bound on the performance of the system when no moment was applied to the foundations. As will be seen further on in this paper, in this case no moment was applied to the footings and the demands on the bearing capacity of the foundations by the earthquake loading were modest.



(a) Elevation of Model



(b) Force-Displacement Response

Fig. 6 Portal Frame Model and associated Force-Displacement Relationship

Table 1. Dynamic Stiffness and Damping Parameter for 3-storey Factor of Safety Design

Footing Group	Dynamic Stiffness Coefficients				Damping Parameters		
	Vertical %	Horizontal %	Rotational %		Vertical (kNs/m)	Horizontal (kNs/m)	Rotational (kNs/m)
1	100	100	99		7.38E+02	1.92E+03	3.46E+02
2	100	100	98		1.55E+03	2.97E+03	8.09E+02
3	100	100	98		1.55E+03	2.97E+03	8.09E+02
4	100	100	97		3.26E+03	4.71E+03	2.23E+03

Table 2. Earthquake Scaling Characteristics

	Fixed Base Period (secs)	Return Period (years)	Scale Factor	PGA (m/s^2)
3-storey	0.863	500	2.161	3.46
10-storey	2.098	500	2.247	3.60

Dynamic Effects

The effect of radiation damping from the footings was investigated by setting-up a model that included the influence of dynamic effects on the stiffness and damping of the foundation soil. Using the methods of Gazetas [19], dynamic stiffness coefficients and damping values were determined for the simple Gazetas spring footing designs. Spring stiffnesses were reduced by the dynamic stiffness coefficients and dashpot members were included in the model to represent the soil damping effects. Again these dashpots members were already available in the RUAUMOKO software. Table 1 indicates the minimal impact on spring stiffness from incorporating dynamic effects in the footing designs considered here.

EARTHQUAKE RECORDS

A single earthquake record was used in the analysis and was applied parallel to the longest plan dimension of the structures. This record was from the La Union event, N85W Michoacan, Mexico 1985. Original records were scaled using the method outlined in the AS/NZS Draft Loadings Standard. Records were scaled to the spectrum representing an earthquake in the Wellington region of New Zealand for a 1 in 500 year return period event. Soil conditions were considered to be Subsoil Class C (Shallow Soil). The characteristics of the scaled earthquake records are shown in Table 2.

RESULTS

Foundation Model Comparison

For the seismic excitation of the 3-storey structure emphasis was placed on determining how the three vertical spring model responded in comparison to the Gazetas spring model. The global response characteristics of each model showed similar characteristics. The response of the footings along the structure parallel to the direction of earthquake propagation changed towards the centre of the structure. Vertical movement of the footings was smaller closer to the centreline of the structure due to larger vertical loads, and the restraint that was provided by the structure above. The footings at the end of the structure had the largest variation in vertical movement.

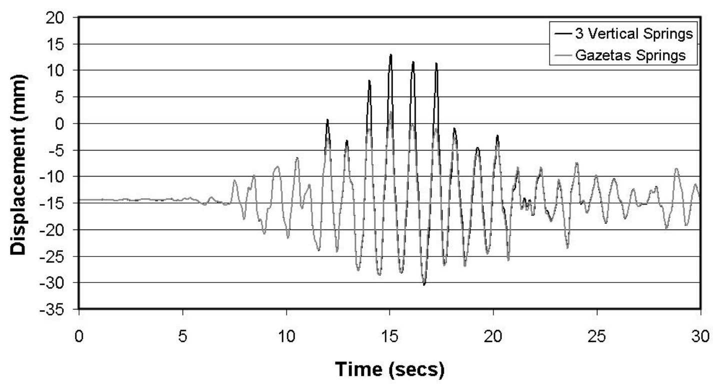


Fig. 7 Vertical Displacement of Corner Footing (Three Storey Structure)

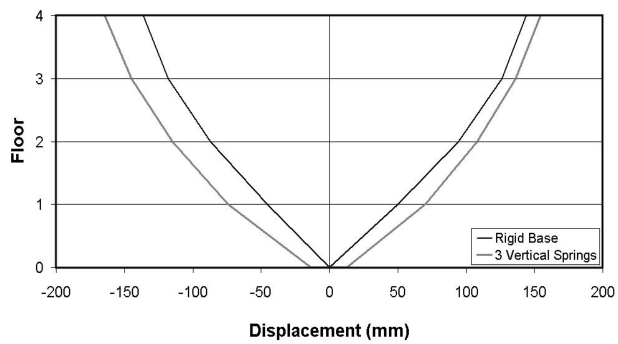


Fig. 8 Peak Structural Horizontal Displacement Envelope (Three Storey Structure)

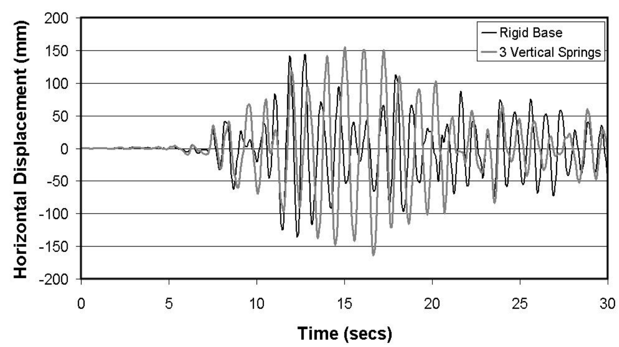


Fig. 9 Horizontal Displacement of Top Floor (Three Storey Structure)

Fig. 7 displays the variation in vertical displacement of the corner footing of the 3-storey structure for the Gazetas spring and the three vertical spring model. Variation in vertical displacement is almost identical for the two models and varies only due to the increased uplift events that occur in the three vertical springs. This difference comes about due to the increased number of vertical springs and the tri- linear force deformation relationship that they develop, similar to the response that occurred in the simple portal frame. Two of the three foundation springs develop uplift at the corner footings and the only vertical restraint provided comes from the final spring which has stiffness that is one quarter that of the spring that is used in the Gazetas spring model. This reduced stiffness leads to the increased number of uplift events, moving the response of the model away from the Gazetas spring model.

Fig. 8 and Fig. 9 indicate the influence that the inclusion of a foundation model has on the response of the structure. This comparison is made between the three vertical spring model and a rigid base, but the comparison could also be applied to the Gazetas spring model as the structural response is similar to that of the three vertical spring model. The horizontal displacement envelope shows that there was an increase in displacement of approximately 20 percent due to the inclusion of the foundation model.

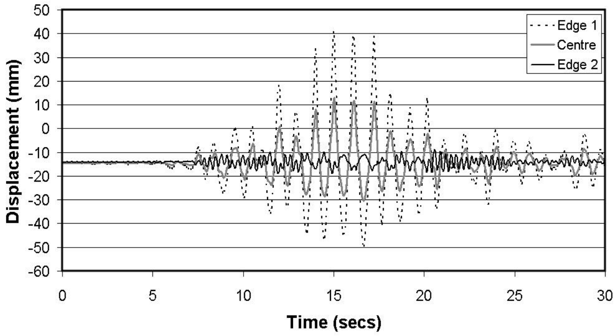


Fig. 10 Corner Footing Vertical Spring Displacement

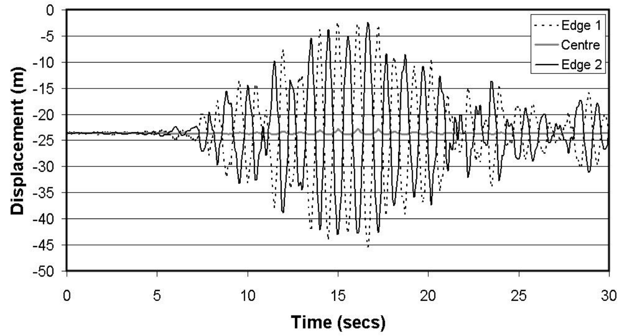


Fig. 11 Footing Number Three Vertical Spring Displacement

Energy Dissipation Mechanism

The manner in which each footing reacted to the seismic loads changed from one footing to the next for the three vertical spring model, and it is this that set the response apart from the Gazetas spring model. Fig. 10 displays the vertical displacement characteristics of each of the foundation springs for a corner footing during excitation. The edge foundation spring that is on the exterior side of the footing had the largest variation in vertical displacement, followed by the central spring. The edge spring that was on the internal side of the structure had a much smaller variation in vertical displacement indicating that the footing seemed to be pivoting about this internal spring.

The other footings in the model did not display this trend. Fig. 11 portrays the vertical displacement characteristics of a footing closest to the centreline of the structure, showing that the edge foundation springs had a much larger displacement than the centre spring. A rocking mechanism developed here to dissipate the moments generated at the base of the column, and there was almost no vertical movement of the central spring. This trend occurred for both the 3-storey and 10-storey structures.

Factor of Safety

Comparison of the footing factors of safety of the two foundation models was made at points of maximum axial force, shear and moment using the Terzaghi bearing capacity equation. At the point in time of each peak action, the other foundation actions were recorded and used to determine the bearing capacity factor of safety. Both models developed similar safety factors at respective points in time but it was the response of the model in comparison to the factors of safety that indicated the accuracy of the model. When the vertical force on a footing reduces, the extent to which the moment reduces the effective contact area of the footing increases, and this should lead to uplift of the footing. The three vertical spring model showed these characteristics as the use of multiple springs allows interaction between moment and vertical force. As uplift of each spring occurred, the vertical and rotational stiffness of the foundation decreased, and this led to a reduction in the moment that the footing carried. Some vertical load is still carried by this footing as one spring was still connected to the footing.

Uplift did not occur in the Gazetas spring model, and the lack of interaction between axial force and moment meant that a larger moment was carried by the footing. The factor of safety calculation indicated that uplift should have occurred but the single vertical spring still carried load and uplift has not been developed. Factor of safety calculations indicated that uplift should have occurred for the outer row of footings perpendicular to earthquake propagation. The three vertical spring model accurately portrayed this, with two springs lifting off beneath the corner footings and one spring lifting off beneath the other end footings.

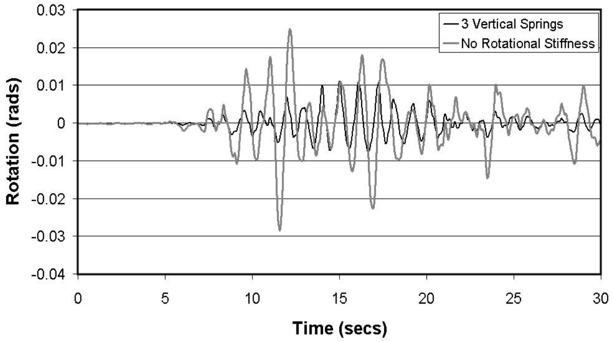


Fig. 12 Rotation of Corner Footing (Three Storey Structure)

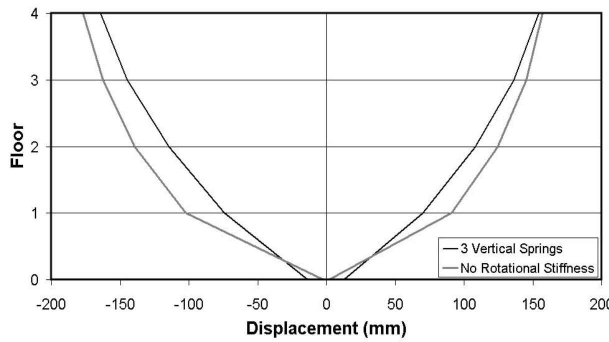


Fig. 13 Peak Structural Horizontal Displacement Envelope (Three Storey Structure)

Footings with No Rotational Stiffness

Comparison was made between the response of the three vertical spring model and the Gazetas spring model without any rotational stiffness to determine the performance of the footings with no moment applied. The lack of rotational stiffness meant that there was a decrease in the vertical and horizontal movement of the footings, while the rotation of the footings increased significantly. The increase in rotation is shown in Fig. 12. As applied moment has the most detrimental effect on the bearing capacity factor of safety of the footings, the safety factors for this model were significantly higher than the three vertical spring model and no uplift occurred.

The lack of rotational stiffness changed the horizontal displacement response of the structure as shown in Fig. 13. Increased rotation at the base of the columns increased the displacement of the first storey of the structure. This increased the interstorey drift of level one, and decreased the drift at higher levels. In Fig. 13 it is apparent that horizontal displacement at ground level was very small when the foundations had zero rotational stiffness. The is because for this case we increased the stiffness of the ground beneath the footings by a factor of 5 (that is Young’s modulus for the soil is $500s_u$, which is 50MPa) as with no moment applied to the foundations the bearing factor of safety decreased little during the earthquake.

Influence of Yielding Structure

The difference in response of the model with a yielding structure was indicated by comparing the response of a 3-storey elastic and a limited ductility structure with three vertical spring foundation model. Yield occurred in the structure before it did in the foundation and this shielded the foundation from the loads developed during seismic excitation. Fig. 14 indicates the reduction in vertical displacement of the corner footings that occurred due to this reduction in force transferral, which was also evident for all the other footings in the model. This led to an increase in the factor of safety of each footing as smaller loads needed to be resisted at footing level. Horizontal displacement of the top floor of the structure also decreased with a limited ductility structure, and Fig. 15 shows this reduced displacement in comparison to the elastic structure.

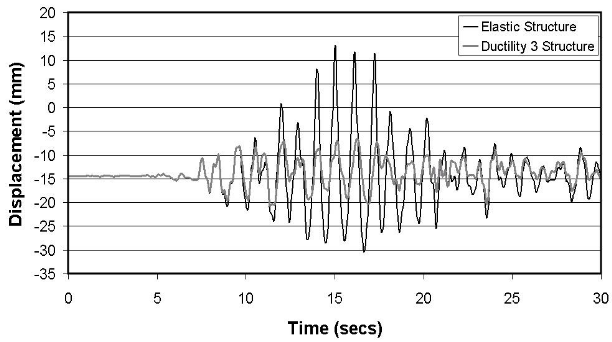


Fig. 14 Vertical Displacement of Corner Footing (Three Storey Structure)

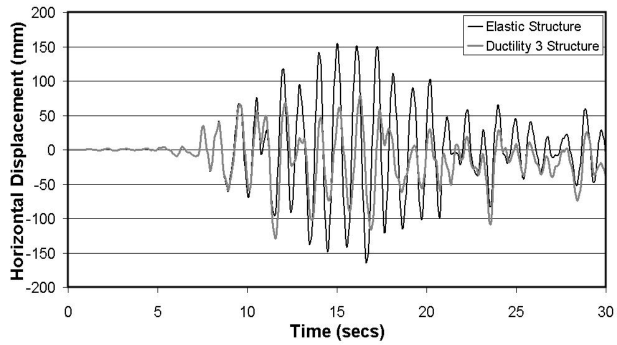


Fig. 15 Horizontal Displacement of Top Floor (Three Storey Structure)

10-storey Structure

Comparisons made above are equivalent to the comparisons that can be made for the 10-storey structure. The major difference between the two models was that the ten storey structure did not develop any uplift during the course of seismic excitation. The reason for this was that the 10-storey structure exerted a much larger static vertical load on the foundations and this produced a static settlement of the footings that was larger than any vertical movement created during excitation. The corner footing has the smallest static load, and as Fig. 16 indicates no uplift events occurred for both the Gazetas spring and the three vertical spring models. As there was no uplift, the springs were still in the elastic range so both had identical vertical displacement traces. This lack of uplift was reflected in the calculation of safety factors, and all footings had factors of safety above unity throughout the duration of excitation.

As with the 3-storey structure the inclusion of a limited ductility structure reduced the actions at foundation level and decreased displacement of the structure. Fig. 17 shows that the elastic structure had peak floor displacement at least 50 percent higher than those developed by a limited ductility structure. This also reduced the peak interstorey drift values throughout the height of the structure.

Horizontal/Vertical Force Comparison

For all models, a check was made on the relationship between the ratio of horizontal and vertical force on the footings and the vertical force to indicate any detrimental impact on the bearing capacity of the footings. The ratio of horizontal to vertical force only increased when the vertical force value decreased, so any reduction in bearing capacity would not have influenced the model characteristics due the smaller vertical loads that were present.

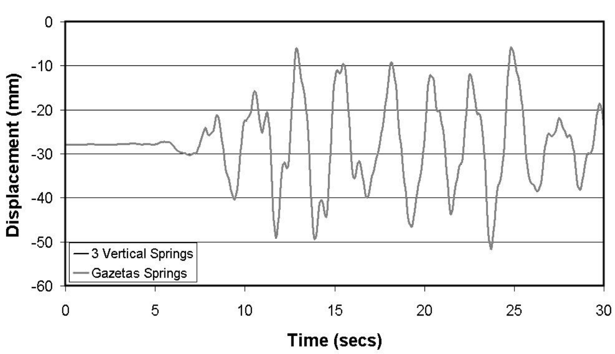


Fig. 16 Vertical Displacement of Corner Footing (Ten Storey Structure)

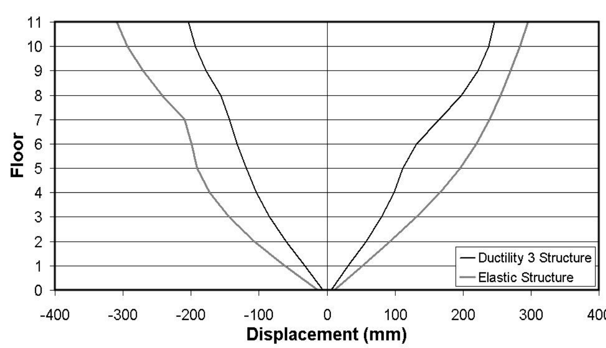


Fig. 17 Peak Structural Horizontal Displacement Envelope (Ten Storey Structure)

Dynamic Effects

The extent of the effect of dynamic factors was determined by comparing models with and without the inclusion of dampers and reduced stiffness. For all models it was evident that dynamic effects had little impact on the response of the structure. There was a negligible decrease in the force and displacement variation with the inclusion of dynamic effects, but nothing that led to a significant change in the response.

FURTHER RESEARCH

The shallow foundation model used herein was very simple, the main thrust of the work being the exploration of the capabilities of an existing software package. We intend to develop the shallow foundation modelling further to have a nonlinear stiffness relation for the footing rather than the bilinear relation used here. Further work will consider the structural models founded on piles. These will be represented using Winkler springs which can yield and also form gaps between the pile shaft and the surrounding soil if required. Another development will extend beyond simple shallow or deep foundations to consider embedded basement structures.

CONCLUSIONS

In this paper we have demonstrated the use of an existing dynamic structural analysis package in modelling the earthquake response of framed structures on shallow foundations. The yielding and uplift characteristics of the foundations have been modelled by adapting available structural models in the software. It is our conviction that the way forward to more effective design of structure/foundation systems is better communication between structural and geotechnical specialists; we see the work outlined in the paper as a contribution towards the achievement of this desirable outcome.

We have looked at both the stiffness of the foundations as well as the capacity. The stiffness was modelled by assuming linear elastic - yielding behaviour of the foundations and the capacity by checking the bearing capacity factor of safety. Specific conclusions from the work are:

- During earthquake loading the shallow foundations are subject to fluctuations about the static vertical load, and also fluctuating horizontal shear and moment. Of these three load fluctuations, it is the moment that places the most severe demand on the bearing capacity of the footings.
- Of the methods for proportioning the footings only that which produced the same vertical stiffness for all the shallow foundations resulted in a satisfactory design. The footings were then sufficiently large to handle the moments generated at the base of the columns for the elastic structure, and the full moment capacity at the base of the columns for the ductile structure. The basis of this design was to proportion the footings supporting the columns in the centre of the structure, which carry the largest vertical static loads but have the smallest vertical load fluctuations during the earthquake, to a static bearing capacity factor of safety of 3.0. The remaining footings were given the same size so that all the footings have the same vertical stiffness. This means that the footings around the periphery of the structure have large static bearing capacity factors of safety but are then able to handle the large fluctuations in load generated during an earthquake.
- The bearing capacity factors of safety were checked at the peak loads on all the foundations. It was found that the lowest values were about 1.3, which is less than the value of 1.8 to 2.0 that has been suggested as appropriate for design procedures that do not employ time history analysis. These values are regarded as acceptable because of the short times for which the bearing capacity factor of safety for some footings is less than 2.0.

- The corner footings for the 3-storey structure were subject to uplift at times during the earthquake loading. In this situation our modelling is slightly unsatisfactory as the models we used did not have a means of detaching the shear and moment springs when a footing lifts off. This problem was alleviated to some extent by our use of a three vertical spring model for the foundations (Fig. 6). Then the discrepancy is not large and occurs only for a short time. The footings of the 10-storey structure were not subject to uplift.
- In view of the above conclusion, there is a need for a more sophisticated shallow foundation model with vertical, horizontal and moment springs linked, so that during uplift the foundation becomes detached from underlying soil and the horizontal and moment springs also detach.
- For the 3-storey structure the assumption of no interaction between the separate shallow foundations is reasonable because of the clear spacing between the foundations. In the case of the 10-storey structure this assumption is not so easily justified because of the reduced clear space between the footings. We have not investigated this further as it is more likely that such a building would have a full basement, which would require a new model. However, one can conclude from the ability of discrete footings to support the 10-storey building that a full basement will have more than adequate foundation bearing capacity.
- RUAMOKO makes it possible to develop an integrated model of the structure/foundation system. A benefit of this is the possibility of investigating the length of time during which the bearing capacity of some of the shallow foundations is low.
- An important outcome of the work reported is the comparison between the behaviour of the elastic structure and the limited ductility structure. The ductile structure yields before the foundation, and acts to shield the foundations. Both the elastic and ductile 3-storey structures exhibited uplift for some footings.

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