Seismic Performance of a Typical Existing Core-Frame RC Building With Soil Struct. Interaction

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SUMMARY:
The seismic performance of a typical reinforced concrete frame - core wall building is investigated, accounting for soil-structure interaction (SSI). The seven story structure is representative of a class of buildings constructed in Greece during the 70s, having relatively long span dimensions and an elevator shear wall core. The building was designed according to the Seismic Design Code in effect using allowable stress design procedures and medium seismicity. The structure was modelled using distributed damage beam-column elements as well as the underlying soil characteristics, introducing rocking and / or uplift of the footings, using equivalent springs as well as a more detailed inelastic model of the core foundation. Static and time history analyses show that depending on the spectral content of the excitation, the use of SSI may lead to both favorable and unfavorable frame response, since it affects the distribution of seismic shears between the core and the frames.

Keywords: Soil-structure Interaction, RC frame, inelastic response, Dual system, Core wall

1. INTRODUCTION

The large inventory of existing reinforced concrete (RC) structures constructed during the 60s and 70s in Greece have been designed following allowable stress provisions for seismic design using the 1959 code (RD59) and using simplified analytical models for structural analysis. In particular, buildings constructed in the 70s in the suburbs of large urban centers in Greece have a regular column layout with a relatively medium span size (5.0 to 6.0m) and are characterized by the presence of a single open cross section shear wall core, intended to house the elevator shaft and stairwell, which is typically located in the centre of the plan. The perimeter frames are infilled with unreinforced masonry walls having continuous openings both horizontally and vertically. Usually, the ground floor is an open storey in these buildings, to house parking or shop areas, depending on their use. The performance of these structures in recent devastating earthquakes in Greece has been primarily characterized by the overstressing and failure of this core wall, being the relatively stiffer element for earthquake resistance, as well as damages at the ground level columns. Due to the lack of ductility and no capacity design provisions in their design, such failures were primarily brittle in nature.

Typically, the perimeter columns were founded near the ground level elevation and were not tied to the wall or to each other using perimeter grade beams or tie beams. Consequently, the relative rotation of the footings and the soil structure interaction (SSI) of these primary earthquake resisting elements with the subsoil – normally considered in design to be fixed at the base –remains to be investigated. Recent analytical studies of the seismic response of simple frames (Dutta et al., 2004, Shakib et al., 2004) have shown that SSI influences the seismic response of rigid systems both in plane and in spatial response, where torsion governs the system behavior. Detailed nonlinear analyses of vibrating footings that develop excessive rotation and toe failure of the ground under extreme seismic events (Anastasopoulos et al., 2010) have shown that soil failure at the toe leading to rotation and footing uplift contributes to a reduction in the seismic excitation of the structure (Gelagoti et al., 2011, Baffo et al., 2007). In order to evaluate in more detail this influence in these typical 70s non ductile RC dual core – frame structures, a typical such building) is reexamined herein, allowing in this case for SSI. The structure was designed under the prevailing codes in the 70s and previously analyzed as a fixed base structure to evaluate its seismic vulnerability (Repapis et al., 2005a, b).

The building is analyzed under static and dynamic analyses: initially an elastic analysis of the building was performed under different foundation conditions, in order to establish the design internal forces (according to the prevailing and the current design loads), to size the footings and to investigate the sensitivity of the former to SSI: four foundation configurations were considered as subsequently explained. Static inelastic analyses are performed using the four models considered, in order to establish the seismic performance of the building following established Static Pushover (SPO) methodologies for the evaluation of existing structures (FEMA 356, KANEPE). Finally, inelastic dynamic analyses of the four building models are performed, under a set of base excitation records suitably scaled to match the spectral design intensities, specified in the response spectra currently in effect.

2. DESCRIPTION OF THE BUILDING AND STRUCTURAL DESIGN

The building considered is a seven storey reinforced concrete (RC) structure that has been typically constructed in major urban centres in Greece for residential construction during the 70s. Following local construction practice during that time, this type of structure is a dual shear wall frame building with a single core U shaped core wall, located centrally in plan to resist seismic lateral loads. The vertical load bearing frame elements have a regular 3.00m storey height and a regular frame plan at 6.0m each way (Fig. 1a). The uniformly distributed loads used for the design of the building, were (Repapis et al., 2005a, b): for the dead loads, in addition to the self weight of the structural elements (including the weight of 16cm thick slabs), a surcharge load of 1.50 KN/m^2 plus an additional 1.00 KN/m^2 for light moveable partitions was considered, together with a line load of 9.0 KN/m on the perimeter beams to account for the double wythe (non structural) perimeter infill weight (namely a design load of 3.60 KN/m^2 of plan façade surface area); for the live load, a load of 2.00 KN/m^2 was adopted in design, typical of residential use. In most cases, the use of perimeter infills was discontinued at the ground storey, to allow for parking space access; however, the contribution of infills to the stiffness of the system was neglected herein.

The building had been initially designed for an allowable stress (service level) seismic zone base shear coefficient of 6% of the building weight, corresponding to a seismic zonation category II (Repapis et al., 2005a). The design material characteristics, following the prevailing code specifications in Greece were DIN B225 concrete (namely C16/20, having average cube strength of 22.5 MPa) and DIN StIII reinforcement (namely S400 according to current specification). According to the design code in effect, service stresses were increased by 30% for the seismic load combinations. Column dimensions were grouped every two floors: interior columns were square, ranging from 60/60 cm for the first two stories, 50/50 cm for the next two, 40/40 cm for the fifth and sixth story and 30/30 cm at the top; perimeter columns were L shaped 25cm wide, ranging in depth from 90cm at the bottom two stories to 70, 50 and 35 cm at the roof. Perimeter beams were 30/60 cm deep while interior beams were 25/50 cm deep, with the exception of the beams inframing into the core wall, which were 20/60cm in size. The core wall, following the typical design practice in the 70s was 20cm thick over the entire height with an internal clear opening 1.8 by 1.8 m. Given the fact that the building was not designed for ductility and special confinement provisions, column stirrup reinforcement consisted of square or rectangular 8 mm diameter stirrups at 30cm spacing; at the core wall corner boundary elements, 8 mm diameter at 25cm spacing stirrups were specified.

A comparison of the internal forces in the core under two different base fixity conditions, resulting from the RD59 (1959) and the currently enforced EAK (2000) design codes, reveals that the core wall was underdesigned for currently enforced seismic design requirements (Fig. 1.b); this can be seen by considering the core capacities at the ground floor, also shown, evaluated according to either Code for the corresponding acting axial load on the core for each case considered (Fig. 1b, dashed lines).

Allowing for the soil flexibility under the core footing (assuming stiff clay) resulted in a reduction of the base shear and bending moment at the top of the footing, which improved the core seismic overstress but it increased, as a consequence, the shear demands in the frames.

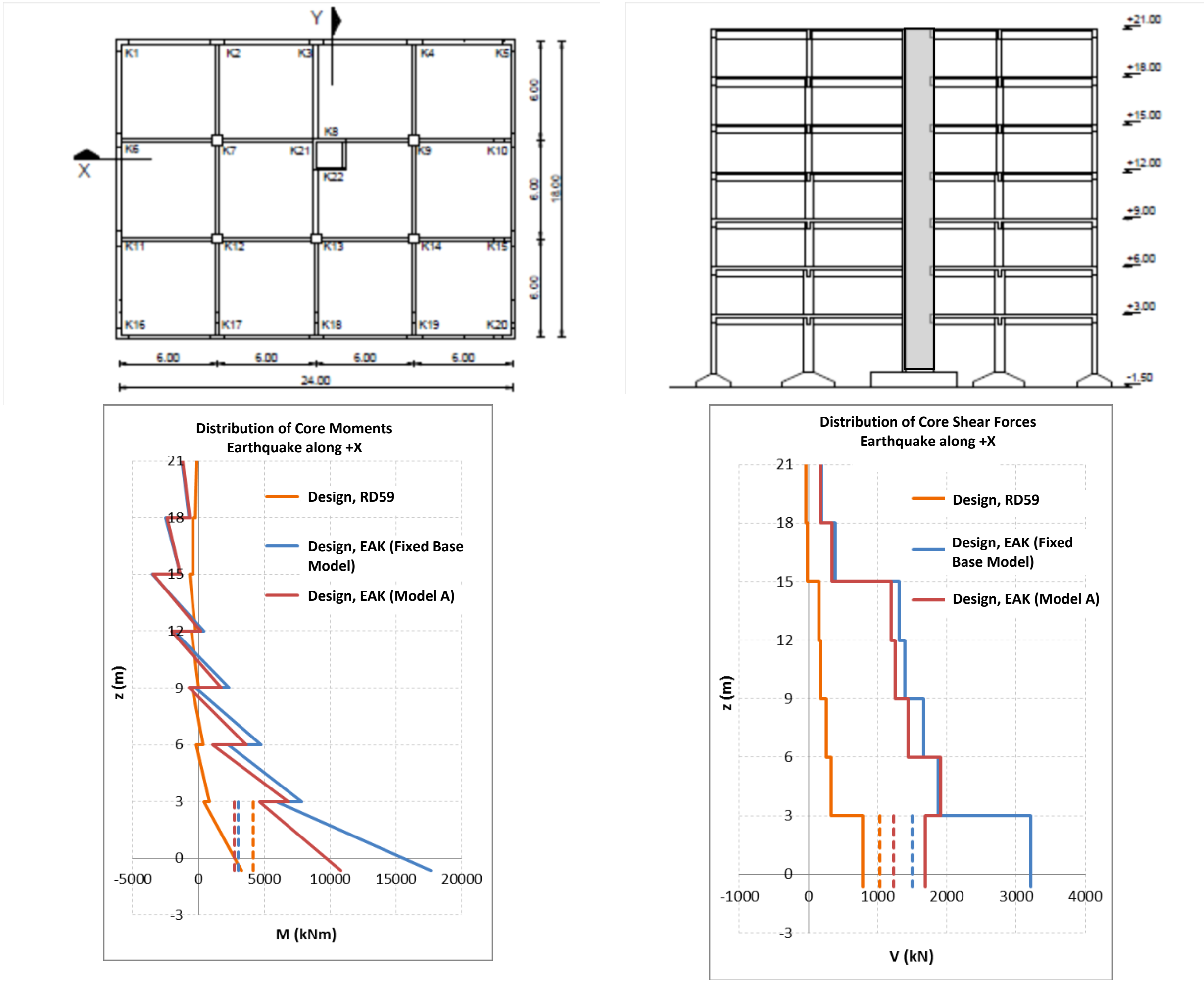


Figure 1. a) Plan and longitudinal section of the subject building analyzed; and b) Comparison of the shear and bending moment design requirements for the core wall, following the RD59 (1959) and currently enforced design provisions (EKOS, EAK, 2000), fixed base and Type A models.

3. DEVELOPMENT OF THE MODEL

The building was analyzed as a three-dimensional frame under initially elastic and, subsequently, Inelastic Static Pushover (SPO) and time history analyses, in the longitudinal direction (the strong axis of the core wall). In all lateral load analysis predictions, a combination of vertical loads equal to the design dead plus 30% of live loads were considered. Two different soil characteristics were considered, namely a stiff clay having a shear modulus G of 60 MPa and a much softer soil, having a shear modulus equal to 20 MPa. Furthermore, three footing configurations were evaluated to reflect the fact that soil conditions were not adequately identified at that time. Consequently, three building – foundation soil types have been considered, in addition to the fixed base structure, as follows:

i) Building/Soil Type A. Footings were designed using an allowable stress approach, assuming a stiff clay with an undrained shear strength S_u = 150 kPa. The soil shear modulus G for the evaluation of the footing springs (as subsequently described) was set equal to 60 MPa. For the evaluation of the footing size, the factor of safety against vertical loads under a seismic excitation combination (FS_v) was selected to be about FS_v = 6.00.

ii) Building/Soil Type B. The soil type was assumed to be similar to Type A (strength and shear stiffness), but the footings were intentionally smaller in size; in this case the factor of safety for the seismic load vertical load combination was selected to be about $FS_v = 2.50$; and

iii) Building/Soil Type C: The soil shear stiffness was assumed to be only 30% of that for Types A and B (namely 20 MPa), while the footings in this case were considerably undersized. In this case, the column footings use a factor of safety equal to $FS_v = 1.50$ while the factor of safety of the core wall footing was kept at 2.50.

The models adopted allowed only for lateral and rotational soil flexibility in the SSI simulations. The differential settlements that would result under the different footing sized adopted was not considered herein. The influence of having flexible footings was established following both fixed base and flexible elastic foundation analyses using SAP 2000 (CSI, 2010). In this case the model was provided with concentrated rotational and translational springs at the core and column supports. The footing rotational and translational stiffnesses were evaluated from the relations proposed for surface rectangular footings under static loading conditions: for a rectangular footing dimension 2B×2L resting on a homogeneous elastic halfspace, these stiffnesses are given by:

Translational stiffness, Y direction: (3.1)

Translational stiffness, X direction: (3.2)

Rotational stiffness, about X direction: (3.3)

Rotational stiffness, about Y direction: (3.4)

The footing dimensions adopted and the corresponding rotational and lateral spring stiffness properties using the expressions above are given in Table 1 for each Soil/Footing type. Furthermore, the initial linear dynamic characteristics of each system are given in Table 2.

Table 1. Building Model Parameters.

Footing	Rotational Stiffness (MNm/rad)			Translational Stiffness (MN/m)		
	Type A	Type B	Type C	Type A	Type B	Type C
Core wall	5484	982	982	756	458	458
Interior Column	1375	635	106	505	389	305
Perimeter column, X	750	431	71	377	315	243
Perimeter column, Y	466	280	43	389	324	251
Corner column	664	172	29	396	251	198

Table 2. Footing Dimensions and Dynamic Characteristics of the Models.

Building Type	Footing Dimensions (m)			T_1 (sec)	Φ_e (m/sec^2)	$\Phi_d = \Phi_e/q$	V_b (kN)	η
	Core Wall	Inter. col.	Corner col.					
Fixed				1.26	3.59	2.11	6891	0.47
Soil type A	5.40/4.60	3.25/3.25	2.55/2.55	1.35	3.43	2.02	6598	0.26
Soil type B	3.00/3.00	2.55/2.55	1.65/1.65	1.41	3.33	1.96	6391	0.11
Soil type C	3.00/3.00	2.00/2.00	1.30/1.30	1.53	3.15	1.85	6050	0.34

The introduction of the partial subsoil flexibility under the footings of the structure resulted expectedly in an increase of the initial elastic period in the fundamental translational mode of the building (T_1) by 22% relative to the fixed base structure (Table 2); this increase in T_1, however, given the design response spectrum shape in this region, did not introduce a significant reduction in the base shear demand of the structure under first mode response – only 12% of the demand for the stiffer fixed base building. What was of paramount importance, however, once the footing flexibility was introduced, was the drastic change in the distribution of the (elastic) design shear forces resisted by the core and the remaining frames. This relative distribution of shears is denoted as η, equal to the ratio of the shear resisted by the core to the total base shear of the building at the footing level (Table 2). Following currently enforced Greek seismic regulations (EAK 2000) this ratio defines the extent to which dual wall-frame systems can be considered to function primarily as a wall system ($\eta > 0.65$) or as a moment frame system that requires, in addition, a joint capacity design in the columns in order to establish a weak beam – strong column behavior. It can be seen then that the more flexible the core footing becomes, the higher the amount of base shear resisted by the frame (between 53% and 89% of the total, depending on the soil / footing properties adopted).

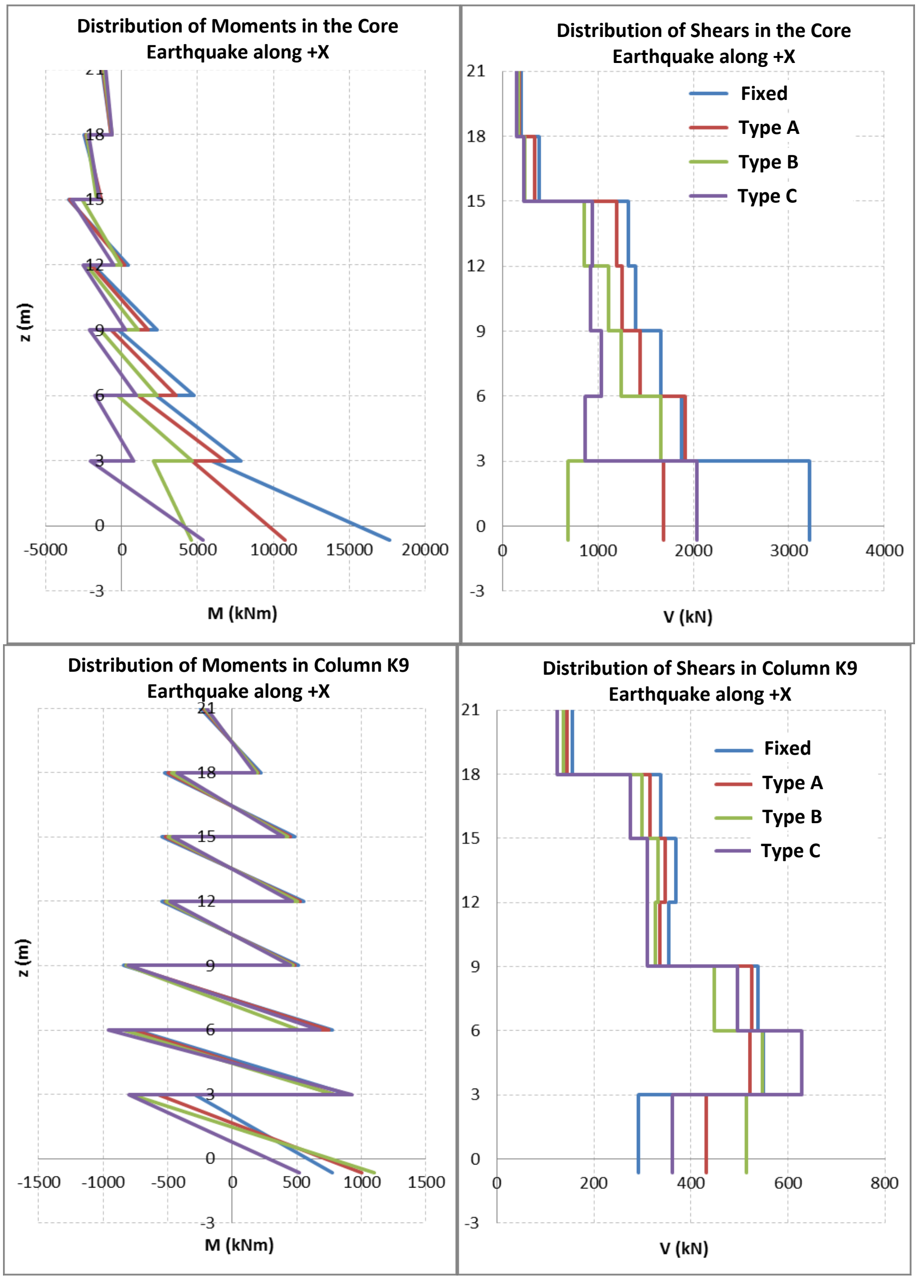


Figure 2. Shear and bending moment distributions for the core wall and neighbouring column K9 to currently enforced design forces (EAK, 2000), for the four models considered

The introduction of SSI in the system resulted in a decrease of the base shear demands and the bending moment distributions in the core, compared to the Fixed Base Model, leading to a corresponding increase to the base storey moments of the neighbouring column K9 (Fig. 2). Considering the distributions with height in Fig. 2, it is shown, under the absence of any higher mode contributions, the influence of SSI to the higher floors of the structure is minimal while all the SSI influences concentrate in the lower stories (both for the core and for the column next to it). Consequently, the peak demands in core shear are incurred in the second floor. Also, the contribution to the seismic shear resistance of the core at the base storey is gradually reduced considerably as the footing becomes more flexible. Exception to this tendency is Model C, where the relative ratio of core wall footing to column footing stiffness is reversed, due to the relatively higher core footing flexibility. In fact, another consequence of the gradual increase in flexibility of the core footing was that the point of zero bending with height (and therefore, the shear span of the core) decreased, making the core more susceptible to a shear dominated bending response than the fixed base model would predict.

Table 3. Static Roof Deformation of the Models and Target Point Displacements for LS (KANEPE 2010).

Building Type	Total roof δ_t (cm)	Footing rotation (rad)	Displacement due to rocking (cm) δ_r	Ratio δ_r/δ_t (%)	Target Point for Life Safety (cm)
Fixed	2.7	-	-	-	28
Soil type A	27.4	0.004	8.8	32	27
Soil type B	28.4	0.009	19.8	70	29
Soil type C	29.9	0.0125	27.1	91	32

Apart from forces, the inelastic roof displacements, evaluated through elastic analysis assuming a response reduction coefficient of q = 1.0, are compared for the four Models in Table 3 and are comparable, with the exception of the Fixed Base model which has a very high stiffness. For the case of the models with flexible base, the relative contribution of the spring rotation at the base to the roof displacement was found to be between 32% (stiff clay model) to 91% of the total roof displacement, for the case of the footings being undersized (Model with Soil type C).

4. INELASTIC ANALYSIS RESULTS

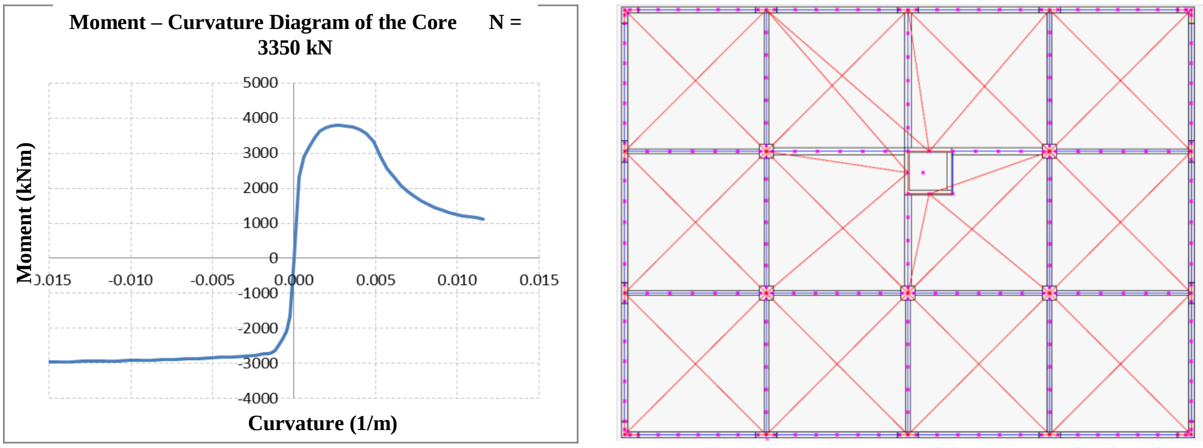


Figure 3. Inelastic moment – curvature characteristic of the shear wall core and diaphragm modelling of the subject building.

A three-dimensional inelastic building model was formulated using line elements with rigid end regions to represent the joints and the core wall – beam connection, using *OpenSees* (McKenna et al., 2007). Material characteristics, loading intensity and section dimension and reinforcement were assumed as already described previously. The actual reinforcement and (confined and unconfined) concrete characteristics of all structural elements (core wall, beams and columns) were modelled using the section fiber discretization and the inelastic force-based beam-column element provided in *OpenSees*, developed by Neuenhofer et al. (1997) (element *nonlinearBeamColumn*). For the concrete materials the nonlinear confined concrete model proposed by Scot at al. (1982) was used, while the steel was assumed to exhibit bilinear kinematic hardening characteristics (characteristic material strengths were used in this analysis, following KANEPE [2010]).

In order to overcome problems associated with modelling the diaphragm using the fiber beam-column elements in *OpenSees*, particularly for the beams inframing to the core, all the beams were assumed to be longitudinally connected through stiff zero length elements (element *zeroLength*) to the column elements, allowing for the transfer of moments only (and not the axial load component) to the joints. Consequently, only the column nodes were connected through the diaphragmatic truss elements to each other. Full diaphragmatic action was simulated using rigid truss elements that connected the columns nodes and the core wall at each floor level. The brace layout in plan is shown in Fig.3.

The inelastic (concentrated) spring characteristics at the base of the core were prescribed also with element *zeroLength* as a generic trilinear moment – rotation relation following the moment-rotation response obtained from a three-dimensional finite element rollover analysis of the footing on a soil halfspace, using Abaqus (Fig. 4). As for the case of the elastic analyses, two different soil models were used, namely the stiff clay of Models A, B (using an FS_v of 6.0 and 2.5) and the soft soil used for Model C. The soil properties (strength, shear modulus and Poisson ratio) were similar to those used for the footing design, while a friction coefficient of 0.7 was used. For comparison, a rigid footing on a Winkler foundation analysis with uplift was also considered (as shown in Fig.4), giving very close results to those obtained from the three-dimensional finite element analysis.

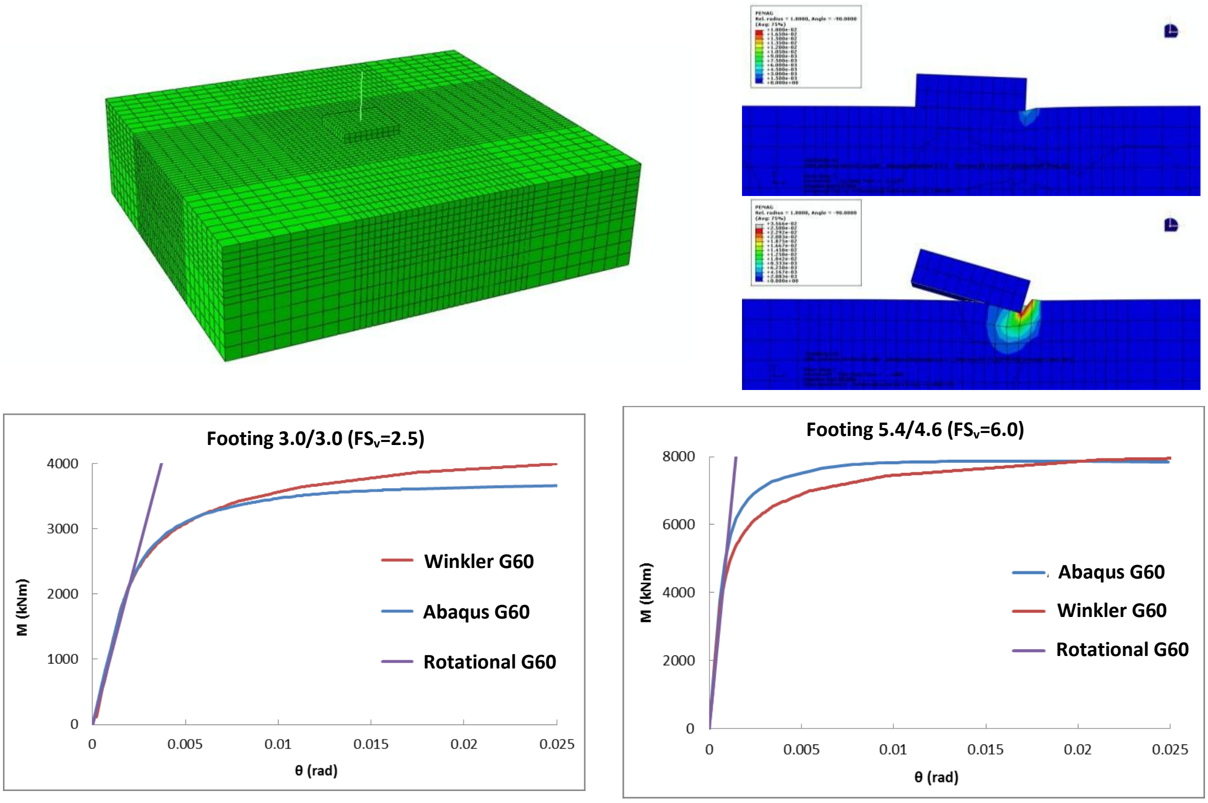
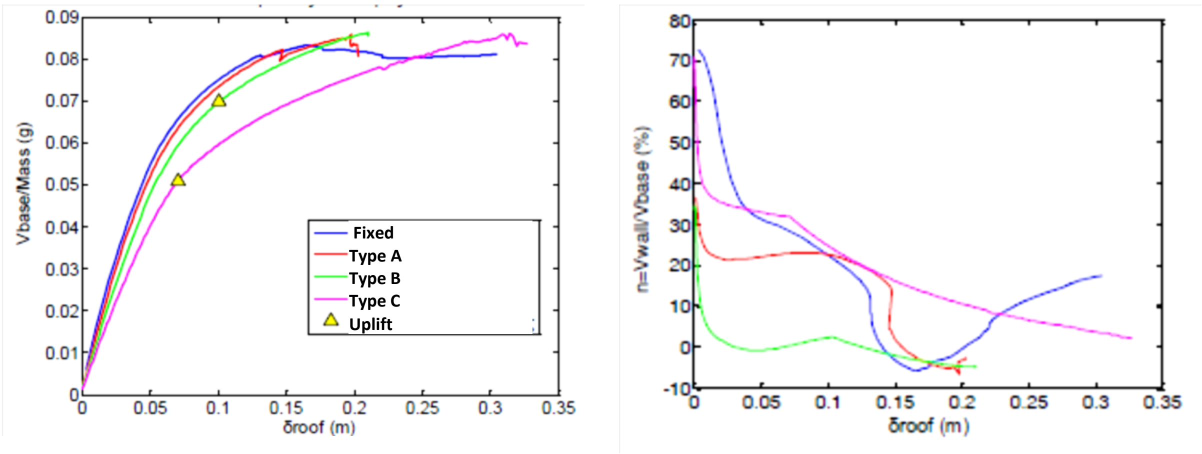


Figure 4. Rollover characteristics of the footings of Models A and B: comparison with a Winkler solution and the rotational stiffness used in elastic analyses

Compared to the core wall moment capacity from the moment-curvature relations of Fig. 3, it is anticipated that for soil Model B (FS_v of 2.5) the footing is unable to resist the peak flexural capacity of the core and will uplift under significant rocking induced rotations. Model A, on the other hand, with a conventionally designed footing, was able to develop the flexural resistance of the core with the footing remaining nearly elastic.

4.1 Static Analysis Results

The SPO analyses results (Fig. 5) indicate that all building models are unable to reach the target point demands for the Life Safety Performance Level, estimated by the Displacement Coefficient Method to be between 27 and 30cm at the roof (Table 3), under the currently enforced Seismic Design Code (EAK, 2000). The critical governing failure for the fixed base model was the core wall failure in shear, while for the flexible footing models failure concentrated in the inframing beams connected to the core, with the core remaining relatively undamaged in this case. Considering the base shear (normalized by first mode mass of the building, equal to 3270 kN/m/sec^2) with roof displacement SPO curves of Fig. 5 it is evident that all four models developed about the same normalised base shear resistance, 85% of the building modal mass. The two models having relatively smaller footings develop uplift of the footing at a roof displacement of 7.0 to 10.cm. Of interest is the variation of ratio η with increasing roof deformation (Fig. 5b). It is seen that the initial elastic predictions of 0.4 to 0.7 (Models B to A and Fixed, respectively), quickly degrade to as low as 0.2 and less as the wall goes into the inelastic range. Evidently, the influence of SSI in the evaluation of the actual distribution of shears in the frame elements is crucial in both the elastic as well as the inelastic range of the response.



(a) (b)

Figure 5. SPO predictions for the four models and footing uplift: a) Base shear versus roof deformation; and b) Relative shear distribution ratio η (in %).

4.2 Dynamic Analysis Results

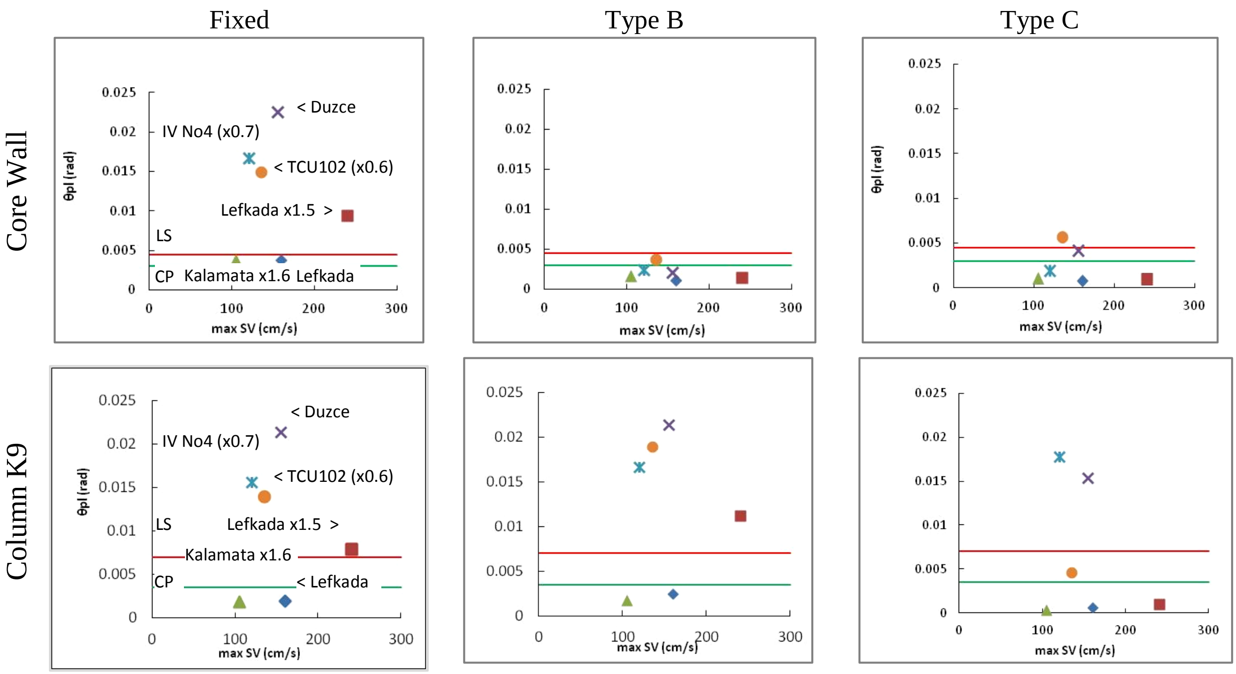


Figure 6. Peak plastic rotation demands at the base of the core wall and at the base of Column K9, next to the core; time history analyses.

The dynamic response of the structural models is evaluated under a set of recorded earthquake excitations normalised to acceleration amplitudes so that the excitation spectral demands were close to the currently enforced spectral design requirements (EAK, 2000), in the vicinity of the first fundamental period of the subject structure (1.0 to 1.5 sec). The ground motions considered were Imperial Valley No4 (scaled to 70%), Lefkada (2003) (at 100% and 150% intensity), Duzce, Chi Chi TCU102 (at 60% intensity) and Kalamata (1985), scaled to 150%.

The peak plastic rotation demands at the base of the core and the neighbouring column K9, resulting from the time history analyses, are compared to the corresponding demands under the Life Safety (LS) and Collapse Prevention (CP) limit states in Fig. 6. Only the three structural models are depicted, since Model Type A behaved fairly close to the fixed base structure. The comparisons of damage indicate that the limit state damage limitations were exceeded in the core primarily in the Fixed Base wall model, due to the significant rocking incurred to the footing of the core under the soft soil assumptions. On the contrary, for the neighbouring column, the presence of SSI led to considerable damages, which were comparable to those incurred at this element under the fixed base assumption, particularly for those base excitations that contained a uniform spectral content (as opposed to a degrading profile) in the period range of the building (e.g., the Duzce, TCU102 and IV No4 records).

Overall, the presence of SSI led to an increase in the fundamental period of the structure, which resulted in a reduction in the spectral acceleration demands but increased the spectral displacements. Consequently, the influence of SSI did not lead to an overall reduction in the structural damages since, depending on the frequency content of the base excitation, it was seen that the columns tended to attract higher shear forces as the core entered in the inelastic range, depending also on the relative rotational stiffness of these elements' footings. Finally, as the case was for the SPO analysis, it was again established that the core wall of the building would not avoid failing in shear at the first storey, under all the base excitations considered.

5. CONCLUSIONS

The objective of the present study was to analytically investigate the seismic performance of existing buildings with a central reinforced concrete core wall (e.g., an elevator shaft), taking into account SSI. For this purpose, a seven storey RC frame-core wall building was examined, which was representative of residential building construction in Greece during the 70s. The structure has a central elevator core as the only available shear wall, concentrating the initial seismic load resistance to this element.

In order to investigate the influence of SSI in the performance of the building under seismic excitation, four cases were considered: the case of a fixed base, Model Type A with a conventional footing design and Models Type B and C, having underdesigned footings, reflecting a choice of a higher allowable stress than actually applicable for the soil at hand. Models B and C have the same footing dimensions under the core wall while Model Type C has smaller footings under the columns. The emphasis of the static and inelastic analyses was to simulate the rotational degree of freedom of the footings and its influence to the overall superstructure performance (drifts, inelastic demands and relative damage distribution between the wall and the columns). In all cases, the vertical component of the foundation deformation was ignored. The behavior of the four structural models was examined using i) elastic static analysis, ii) SPO inelastic analyses and, iii) primarily inelastic dynamic analysis. The results of the study, lead to the following conclusions:

- The effect of including SSI in the model was to reduce the relative contribution of the core in seismic load resistance, leading to a higher participation of the frames of the dual system in resisting the earthquake. Consequently, the fixed base structure and Model Type A having a stiff clay foundation, resulted in the highest bending moment and plastic rotation demands for the core. More flexible structural models (due to SSI) resulted in higher internal forces to the frame members. Models B and C exhibited core footing uplift under rocking prior to the core developing its ultimate flexural resistance and consequent reduction in core demands. The internal forces resisted by the columns depended on the stiffness of the column footings relative to the core footing. Model Type C, with smaller footing dimensions for the core and, thus, a relatively stiffer column footing, behaved better, by allowing both the core to rock and the stiffer columns to absorb higher forces.

- The inelastic dynamic response of the structure depended strongly on the frequency content rather than the peak ground acceleration of the base excitation. Excitations with decreasing spectral contents with increasing period resulted in reduced plastification of the frame. Where excessive plastification was induced in the elements, the first fundamental period increased by over 100%.

- The reduction of the lateral stiffness at the base of core either due to plastification and/or due to SSI, resulted in a reduction in base shear resisted by the core. For the given wall – frame configuration the peak shear force was developed between the 2nd and 3rd stories. Currently enforced design provisions for dual frame buildings would underestimate the shear forces at these levels; more importantly, the inelastic distributions of bending moment and shear force in the core deviate considerably from those prescribed in current capacity design requirements for dual systems, for designing the wall (particularly the shear span ratio, affecting the shear resistance estimation of this element).

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