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THE ROLE OF SOIL-STRUCTURE INTERACTION IN THE REDISTRIBUTION OF INTERNAL FORCES IN CONCRETE BUILDINGS DUE TO SEISMIC EXCITATION

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Abstract. *This work studies the role of soil-structure interaction in the redistribution of internal forces among the various vertical load bearing elements of concrete buildings due to seismic excitation. According to the Greek and European Design Codes the columns are designed assuming a fixed base. However, as it is shown in this work, under certain conditions, this is not the critical design state. Soil-structure interaction could lead to redistributed internal forces considerably higher than those predicted under the fixed base assumption. For the purposes of this study a number of "common" simple 3-D concrete frames and their foundations are designed and analyzed according to the specifications of the Greek Earthquake Design Code. Subsequently the foundations are substituted by a system of springs, dashpots and added masses, simulating the phenomenon of soil-structure interaction and the frames are re-analyzed. In both cases, i.e. fixed base or soil-structure interaction, the structures are subjected to artificial seismic motions consistent with acceptable design spectra. Comparison studies between the two assumptions are conducted and conclusions with practical interest are drawn.*

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1 INTRODUCTION

According to the Greek Design Codes [1,2] and the equivalent European Codes, e.g. EC8 [3], the analysis and design of structures, especially buildings, due to seismic excitation, should be implemented under the assumption of complete fixity at the base of all vertical elements. This assumption is considered to be on the safe side since, according to this line of thought, it causes maximum internal forces in the vertical load bearing elements. Subsequently the foundations are designed for these internal forces, without taking into consideration the interaction between the soil and the structure. In this work it is shown that, under certain conditions, this assumption does not lead to a critical design state.

In general, soil-structure interaction (SSI) is divided into two components, the kinematic interaction, which is significant in deep and embedded foundations, and the inertial interaction which is always present. In this work only surface foundations are analyzed and, thus, the investigation is limited to inertial interaction. In these cases SSI causes a, usually small, increase, as compared to the fixed base model, of the basic modal period(s) of the structure. Taking under consideration the design spectra used by most design codes, i.e. EC8 [3], Greek Seismic Design Code [2], etc., this usually leads to a reduction of the base shear force while the overall displacements are increased. If the foundation system (soil and structure) of the building is stiff enough, the distribution of internal forces among various vertical elements remains close to that predicted by the fixed base assumption. However, if the soil flexibility allows appreciable deformation under individual footings, in the presence or not of tie beams, then a redistribution of internal forces takes place (as compared to the fixed base assumption), which may lead to a substantial increase of internal forces to some vertical elements and reduction to others, while at the same time the total base shear is actually reduced. In order to specify and quantify the role of SSI in this phenomenon, the following methodology is used.

A. Simple, 3-D concrete framed structures are designed so that they are marginally adequate according to the requirements of the Greek Design Codes [1,2]. The foundation consists of independent footings, under each vertical element, connected with tie beams. The above structures, under the fixed base assumption, are subjected to a seismic motion in the form of an artificial accelerogram, consistent to code specified design spectra [2]. Linear time step analyses are performed to each of these structures and the maximum internal forces of the vertical elements are recorded.
B. In order to take under consideration the deformability of the soil and the SSI effect, the previous fixed foundations are substituted by a system of frequency independent springs, dashpots and lumped masses. The produced structural systems are subjected to the same seismic actions (artificial accelerogram) and the internal forces of the vertical elements are recorded again.
C. Finally the results of the above two analyses, with and without SSI, are compared and conclusions are drawn.

2 METHODOLOGY

2.1 Design of the structures

Fifteen simple structures have been designed, in order to be marginally adequate according to the requirements of the Greek Design Codes [1,2]. Most of them are one-storey structures with the storey high kept constant at h=3m, in all cases.

The internal forces of all vertical elements due to seismic excitation, are computed using modal response spectrum analysis and the CQC method. The elastic pseudo-acceleration design spectrum, as specified by the Greek Earthquake Design Code [2], is used, where the importance factor (γ_I), the foundation factor (θ) and the damping correction factor (n) are equal to unit. The spectrum amplification factor (β_0) is equal to 2,5 and, assuming soil category C with average wave velocity v_s=200m/sec, the characteristic periods T_1 and T_2 are 0,20sec and 0,80sec respectively. The peak ground acceleration (A) is assumed at 0,36g while the behavior factor (q) is taken as unit, since linear analyses are conducted.

The main code requirements can be stated briefly as:

a) Limitation of interstorey drift Δ should be limited as

$$\begin{gathered} \frac{\Delta}{h} \leq 0{,}005 \\ \frac{N_{st}\Delta}{V_{st}h} \leq 0{,}10 \end{gathered} \tag{1}$$

where N_{st} and V_{st} are the storey axial and shear forces, respectively.

b) Resistance of vertical elements to shear force and biaxial bending with axial force.

In order to achieve a marginally adequate structure the storey mass and the reinforcement of the vertical elements is altered until all vertical elements acquire a capacity ratio (C.R.) near to unit. In most cases the critical requirement is the resistance to biaxial bending with axial force. Subsequently the required foundations are designed according to the usual fixed base assumption [1].

2.2 Soil-Structure Interaction SSI

The interaction between the soil and the structure is considered via a frequency independent mass-spring-dashpot model representing the interaction forces at the foundation-soil interface. Among the various available such models, simulating a linear elastic, homogeneous, isotropic half space, the one proposed by Mulliken & Karabalis [4] is used in this work and is reproduced in Table 1 for easy reference.

	Equivalent half width, a	Equivalent radius, r_0	Mass (inertia) ratio, β	Soil mass (inertia), m_v	Stiffness, K	Damping, C
Vertical	$0{,}5\sqrt{l_x l_y}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{1-v}{4}\frac{m}{\rho r_0^3}$	$\frac{0{,}27m}{\beta}$	$\frac{4{,}7Ga}{1-v}$	$\frac{0{,}8a}{V_s}K$
Horizontal	$0{,}5\sqrt{l_x l_y}$	$\frac{2a}{\sqrt{\pi}}$	$\frac{7-8v}{32(1-v)}\frac{m}{\rho r_0^3}$	$\frac{0{,}095m}{\beta}$	$\frac{9{,}2Ga}{2-v}$	$\frac{0{,}163a}{V_s}K$
Rotational about x	$0{,}5\sqrt{l_x l_y^3}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{3(1-v)}{8}\frac{m}{\rho r_0^5}$	$\frac{0{,}24m}{\beta}$	$\frac{4Ga^3}{1-v}$	$\frac{0{,}6a}{V_s}K$
Rotational about y	$0{,}5\sqrt{l_x^3 l_y}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{3(1-v)}{8}\frac{m}{\rho r_0^5}$	$\frac{0{,}24m}{\beta}$	$\frac{4Ga^3}{1-v}$	$\frac{0{,}6a}{V_s}K$
Torsion	$\max\begin{Bmatrix}0{,}5\sqrt{l_x l_y^3}\\0{,}5\sqrt{l_x^3 l_y}\end{Bmatrix}$	$\frac{2a}{\sqrt[4]{3\pi}}$	$\frac{m}{\rho r_0^5}$	$\frac{0{,}045m}{\beta}$	$8{,}31Ga^3$	$\frac{0{,}127a}{V_s}K$

Table 1: Mass, stiffness and damping coefficients

For each degree of freedom of the rigid rectangular foundation the adopted model consists of a lumped mass *M* which is the sum of the total mass (or rotational inertia) of foundation (*m*) and an interacting soil mass (m_v), a spring with stiffness *K* and a dashpot with constant *C*. The produced mass, spring and dashpot constants for the six possible degrees of freedom of each foundation are independent of each other, i.e. relaxed boundary conditions are assumed at the soil-foundation interface.

In Table 1 the values of the proposed coefficients for mass, stiffness and damping coefficients are shown, for a rectangular foundation with sides l_x and l_y. The symbols G, ρ and V_s stand for the shear modulus, mass density and shear wave velocity, respectively, of the soil medium.

2.3 Linear time step analysis

Although the structures used in this work are designed on the basis of the specified design spectrum and the usual design code procedures [1,2] outline previously, the quantities of interest, i.e. internal forces and moments of vertical elements, are obtained via time domain analysis. For this purpose an artificial accelerogram, consistent with design spectra, is produced by processing a random signal in time and frequency domain, with duration of 30 sec. This artificial accelerogram is amplified by 4,4% in order to reach the design code requirement (E_x±0,3E_y) for a combination of seismic actions in two orthogonal horizontal directions. The same seismic action is applied to all structures analyzed in this work, regardless of the boundary condition at the base, i.e. fixed-base or SSI, in eight distinct directions as shown in Figure 1.

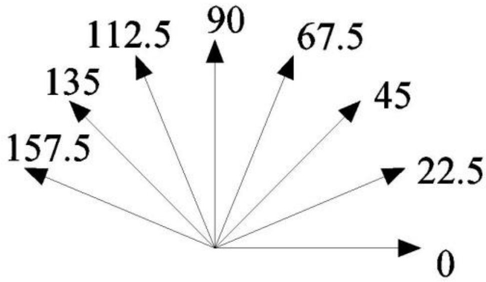

Figure 1: Directions of seismic excitation

The structures are modeled by usual beam and shell finite elements and masses lumped at the nodal points of the finite element mesh. A damping matrix [C_{sup}] proportional to the stiffness matrix [K_{sup}] of the fixed-base superstructure is also assumed. The coefficient of proportionality b, defined in the usual way as

$$b = \frac{\zeta T}{\pi} \tag{2}$$

where ζ=0,05 and T is a weighted average of the mean periods of the structure in two orthogonal directions x and y which are given by

$$T_i = \frac{\sum_{n=1}^{N} T_n M_n^*}{\sum_{n=1}^{N} M_n^*} \quad i = x, y \tag{3}$$

whith the modal periods T_n and effective masses M_n^* computed in the two orthogonal directions of seismic action. The Newmark average acceleration method is used for the computation of the time domain response of each structure and the identification of the maximum internal forces for each vertical element: shear forces in each direction and the capacity ratio in biaxial bending with axial force. Also the maximum total base shear forces in each direction are recorded.

The above procedures are applied to the same structures either under fixed-base conditions or considering the interaction with the soil. Finally, the maximum internal forces recorded in each structure, under both boundary conditions, are compared and conclusions are drawn.

3 NUMERICAL RESULTS

In Figures 2 to 16 the structures designed and analyzed in this work are shown. On the same figures the results obtained from the comparison between the fixed-base and SSI boundary conditions are recorded. Thus, next to each vertical element three coefficients are shown, namely: V_x, V_y and M-N, which reflect, respectively, the variation of shear force in the directions x and y and the capacity ratio in biaxial bending with normal force. Each coefficient is the ratio of the maximum quantity recorded from the analysis with SSI to the respective maximum force from the analysis of the same structure with a fixed-base. In addition, the maximum M-N capacity ratio, of the SSI case only, is shown in brackets, as an indication of the actual structural sufficiency (or insufficiency) of the corresponding vertical element. It should be noted that at the design stage the M-N capacity ratio is kept as close to unity as possible.

The coefficients V_x, V_y shown at the top of each figure designate the ratio of the maximum recorded total base shear force in each direction when SSI is considered to the same maximum quantity when a fixed base boundary condition is implemented..

Structure 1, shown in Figure 2, is a simple, one storey, square structure with basic modal period 0,21sec, when the base is fixed. As expected, a reduction of base shear force appears which is uniformly distributed to all four columns.

Structure 2, in Figure 3, is also a one storey, square structure with eccentricity in the x-axis equal to 25% of the respective length of structure. The basic modal periods are T_x=0,21sec and T_y=0,19sec, for the fixed-based structure. A reduction of base shear force appears in both directions but, as it is expected, it is not distributed uniformly among all columns. Stiffer columns are relieved more than the less stiff. The reduction of the base shear in the x direction is 13%. The respective shear of stiffer columns reduces by 16%, and of the less stiff columns by 9%. The same behavior is observed in the y direction as well, with a reduction of total base shear force reaching 17%, stiffer columns shear 20% and less stiff columns 14%.

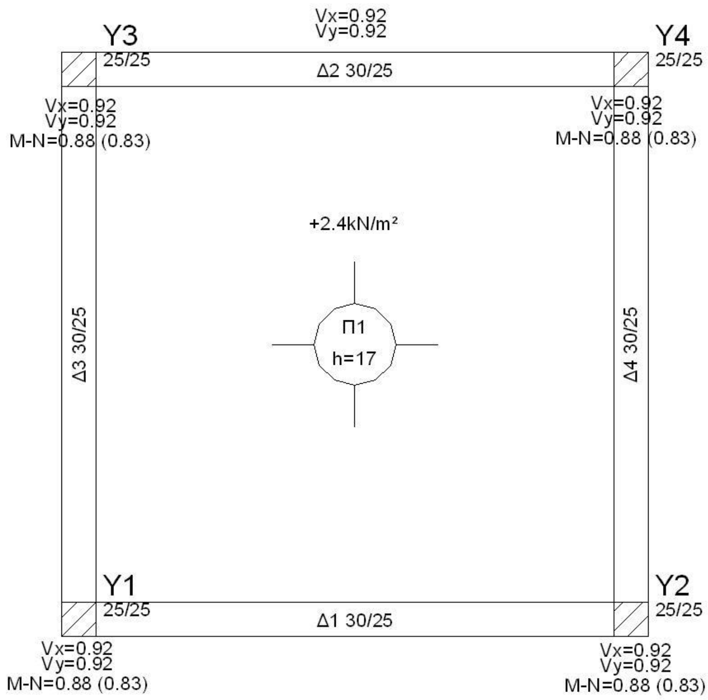

Figure 2: Structure 1

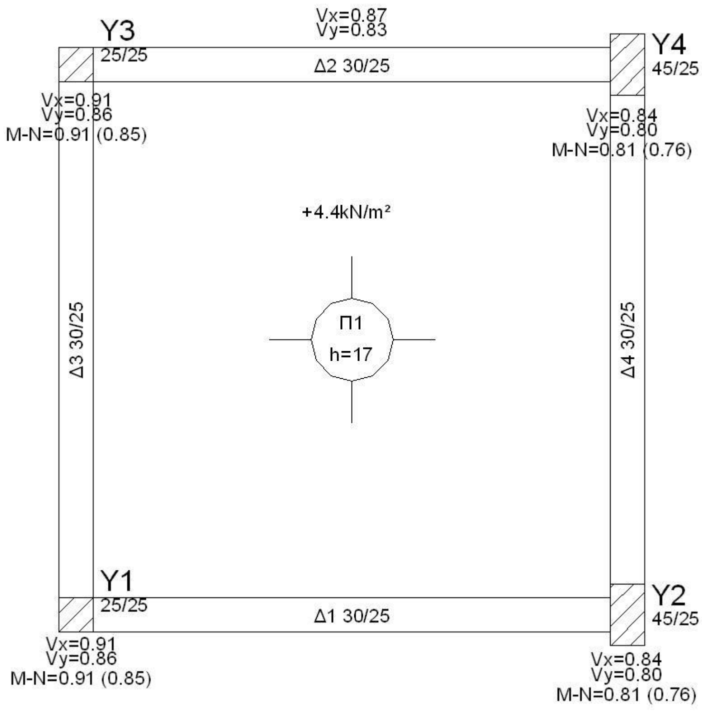

Figure 3: Structure 2

Structure 3, in Figure 4, is similar to structure 2 but with an eccentricity of 42% of the respective length of the structure. The basic modal period in x direction is 0,20sec and in y direction is 0,17sec, for the fixed-base structure. An increment of 16% of base shear force in the y-direction, compared to the fixed-base structure, appears when SSI is considered. This was expected since the basic modal period of the fixed-base structure is less than 0,20 sec. In the x-direction a further reduction of base shear appears. In both directions the variations in base shear are not distributed uniformly, as the stiffer columns are more relieved than the less stiff ones. The increase of base shear by 16% in y-direction results in an increase of the respective shear force by 24% in less stiff columns, and by only 11% in stiffer columns. The reduction by 22% of base shear force in x direction causes a 30% reduction to stiffer columns and only 3% reduction to less stiffer elements. All columns are acceptable according to the M-N criterion.

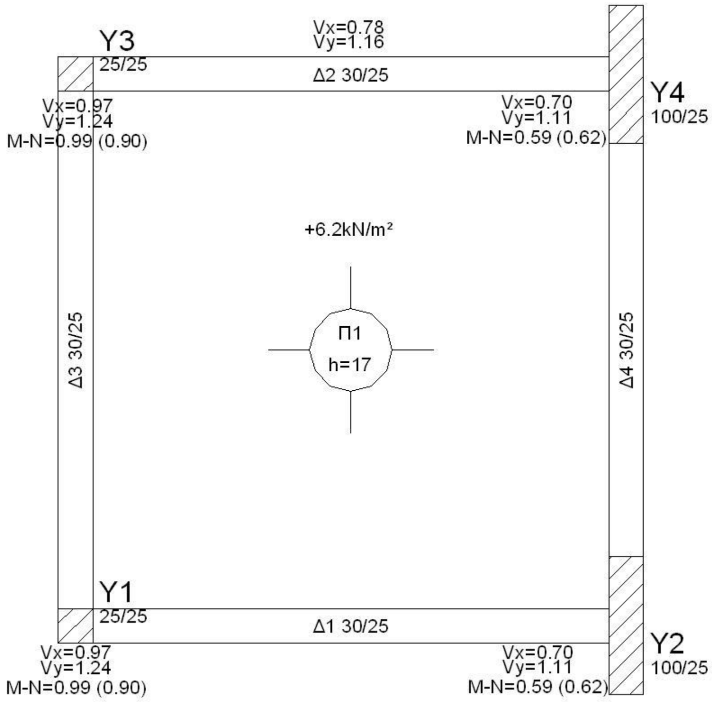

Figure 4: Structure 3

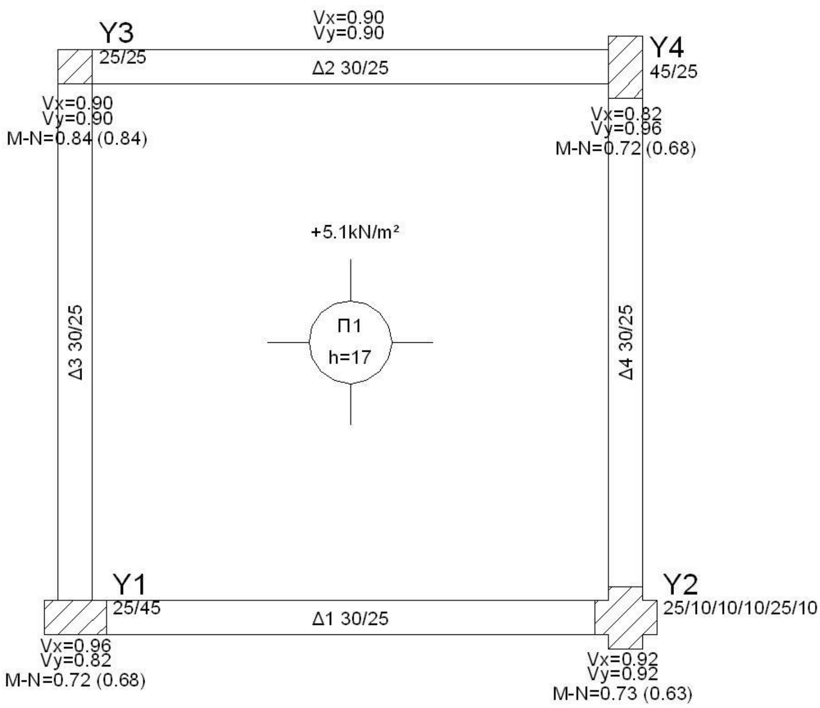

Figure 5: Structure 4

Structure 4, in Figure 5, has eccentricity of 22% of the respective length of the structure, in both axes. The basic modal period of fixed structure in both directions is 0,17sec. A reduction of base shear force in both directions appears when SSI is considered, as compared to the fixed-base condition, but again it’s distribution is not uniform. Still all vertical elements are sufficient according to the M-N criterion.

In structure 5, in Figure 6, a larger eccentricity, equal to 40%, in both directions, is applied and as result the total base shear increases at 42%, compared to the fixed-base condition. This is mainly due to the increased overall stiffness of the structure, which is indicated by the lower basic modal period of 0,11sec. However, the distribution of the base shear changes in a non-uniform manner with less stiff elements receiving most of the change (as compared to the fixed base condition) and even becoming insufficient according to the M-N criterion.

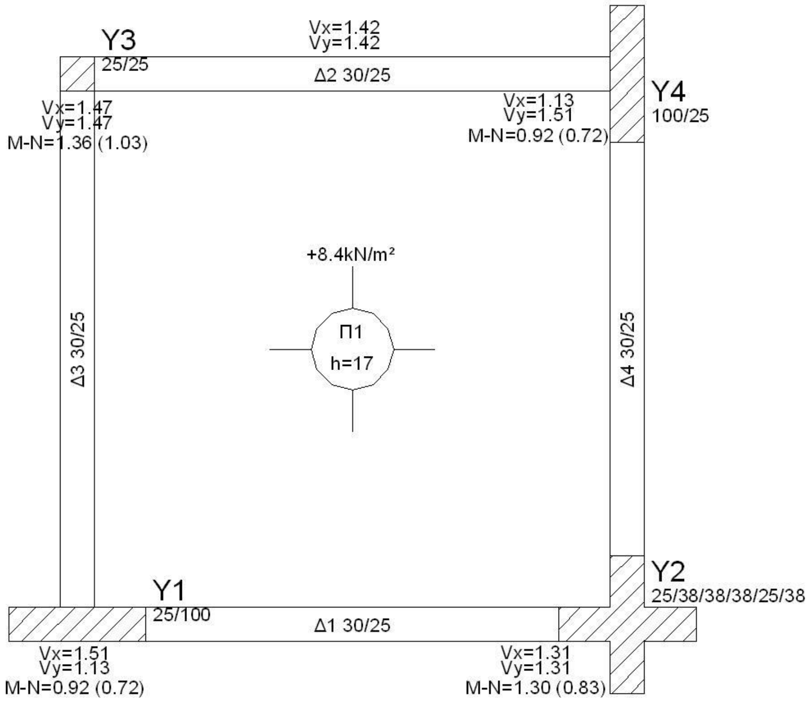

Figure 6: Structure 5

From the analyses of structures 1 to 5 it is not clear if the geometrical irregularity (eccentricity) or the differences in stiffness among the various vertical elements or both is responsible for the redistribution of the internal forces. So the next two structures, i.e. structure 6 in Figure 7, and structure 7 in Figure 8, have the same eccentricity in one direction (17%) and the same basic modal periods (0,17sec in x and 0,20sec in y), but in structure 6 this is achieved with three stiffer columns in one side while in structure 7 with one stiffer column.

In structure 6 a small reduction of base shear forces appears (5% in x and 1% in y), but in structure 7 an increase appears especially in the strong direction (12% in x and 5% in y). In both structures the phenomenon of the stiffer elements being relieved, when SSI is present, is apparent, while at the same time the weaker elements are further stressed. However, this is more intense in structure 7 where the shear force relief of the strong element is 11% and at the same time its neighboring elements experience substantial increases of up to 79%. Thus, it seems that the determining factor for the redistribution of shear force is the differences in stiffness between vertical elements. The stiffer elements are relieved while the highest increases appear in the elements connected to them. In addition, column Y5, directly connected to the strong column Y2 of structure 7, became insufficient in view of the M-N criterion.

Structure 8, shown in Figure 9, is designed with a single stiffer element at its center which receives 30% of the total base shear force, under the assumption of a fixed base. Similarly structure 9, in Figure 10, has an even stiffer element at the same location which receives 50% of the total base shear force. In both cases the total base shear force presents a minor change when SSI is considered. However, due to the redistribution of shear forces the stiffer central columns of structures 8 and 9 are relieved by 13% and 32%, respectively while the largest increases occur at the columns directly connected to the stiffer central column which undertake 25% and 43%, respectively, more shear force under SSI conditions than that under a fixed-base assumption. All these phenomena are, of course, more intense in structure 9, in which the difference in stiffness between the vertical elements is more pronounced. Furthermore, all the columns in the periphery of structure 9 connected to the central column become insufficient for combined biaxial bending and axial force (M-N criterion).

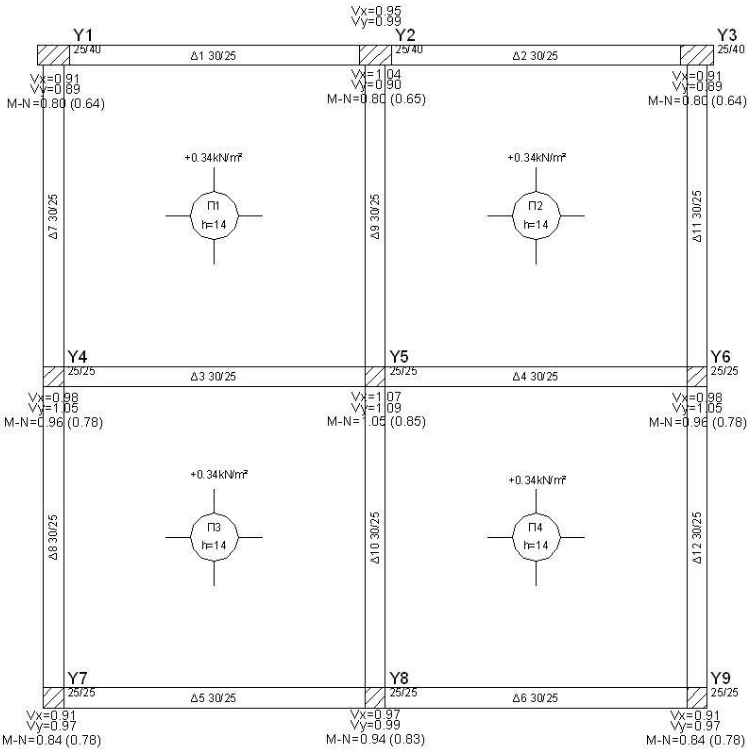

Figure 7: Structure 6

In contrast to structures 8 and 9, structures 10 in Figure 11, and structure 11 in Figure 12, are designed with the stiffer vertical elements in their perimeter, following the usual code suggestions. Under the fixed-base assumption these stronger elements undertake 30% and 50%, respectively, of the total base shear force. In both cases the total base shear force remains virtually unchanged under any boundary condition assumption at the base of the structures.

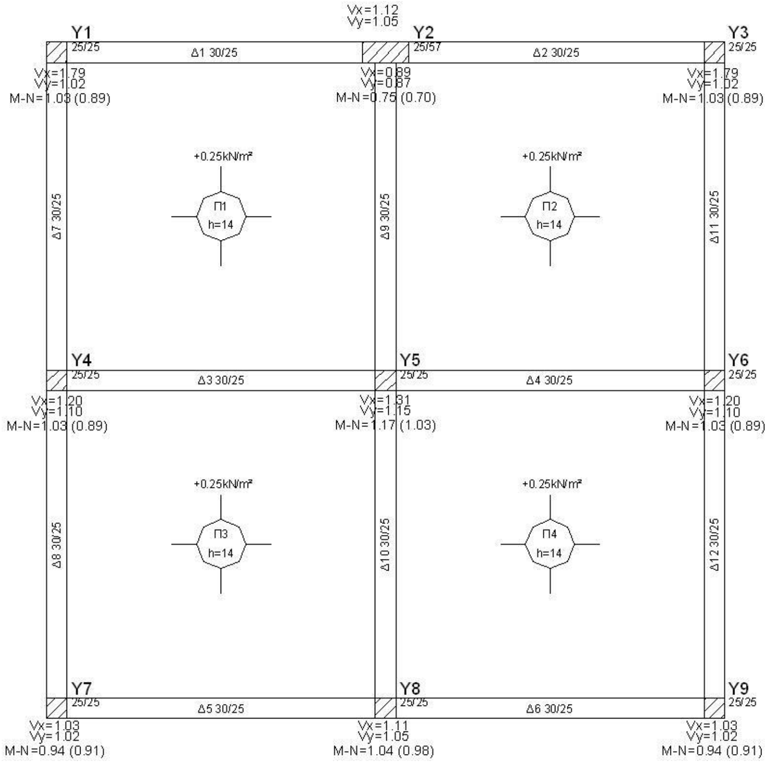

Figure 8: Structure 7

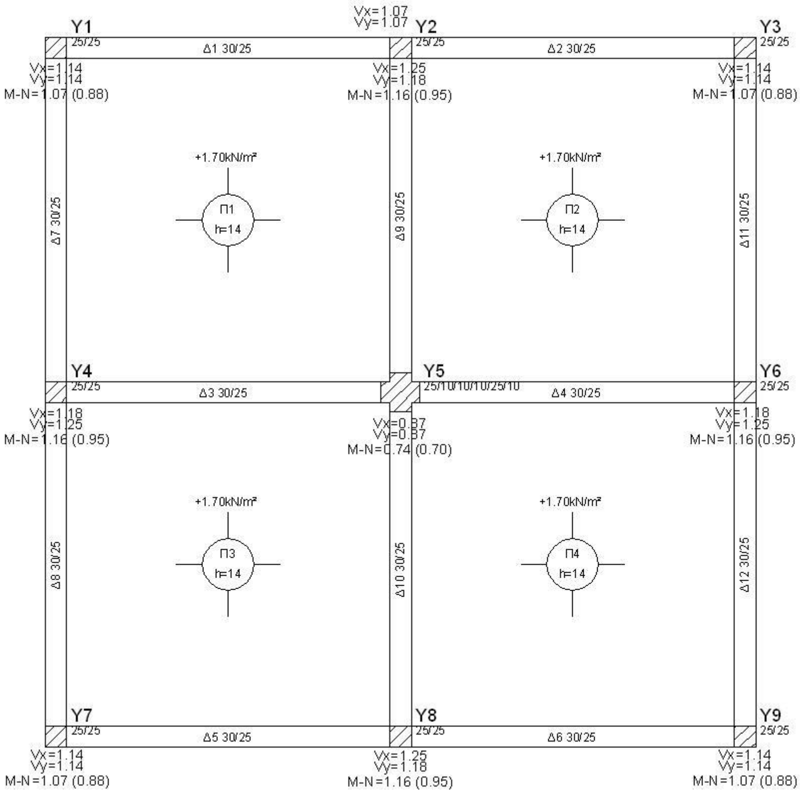

Figure 9: Structure 8

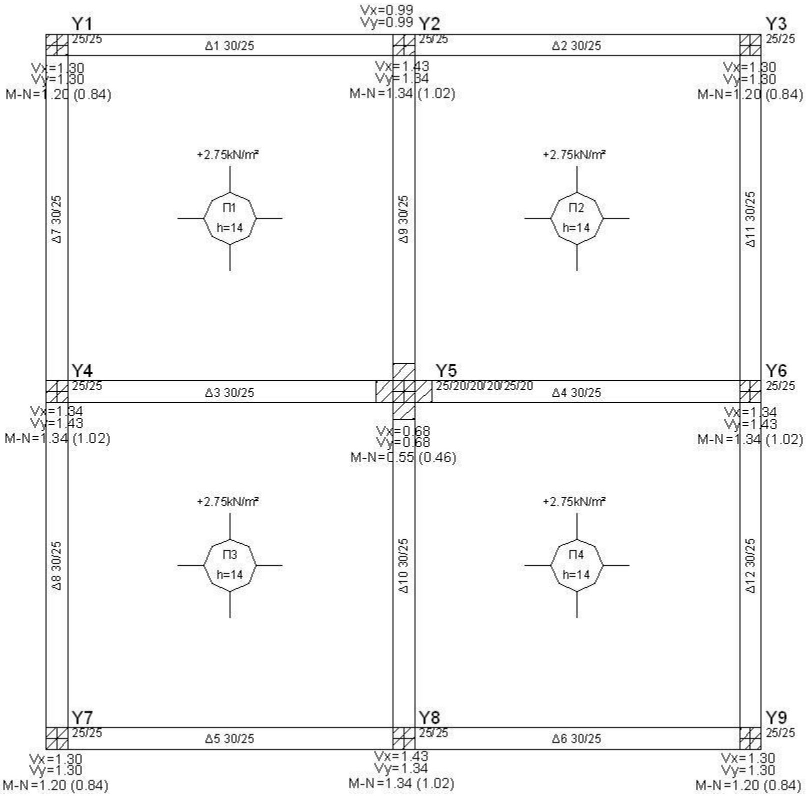

Figure 10: Structure 9

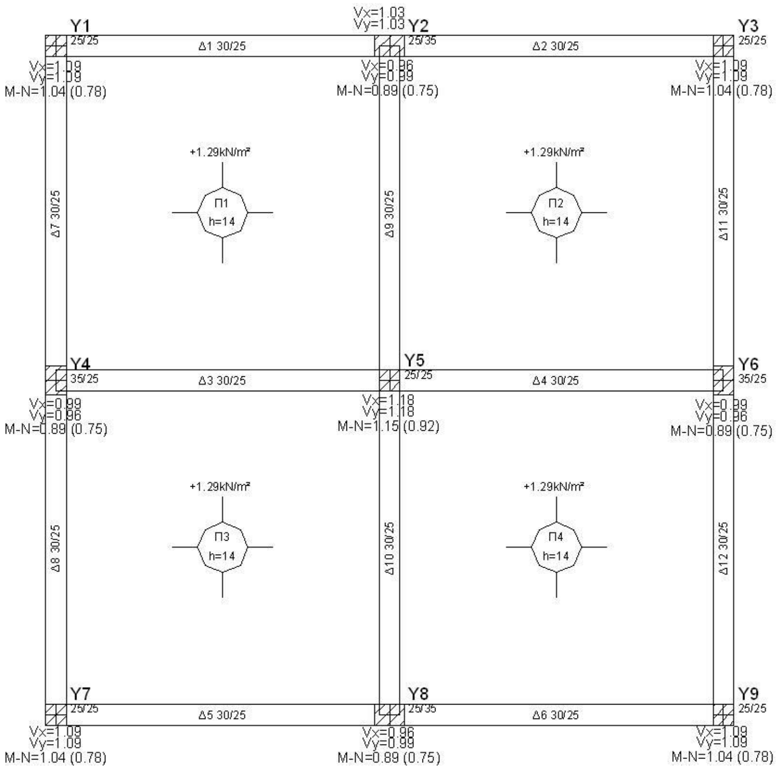

Figure 11: Structure 10

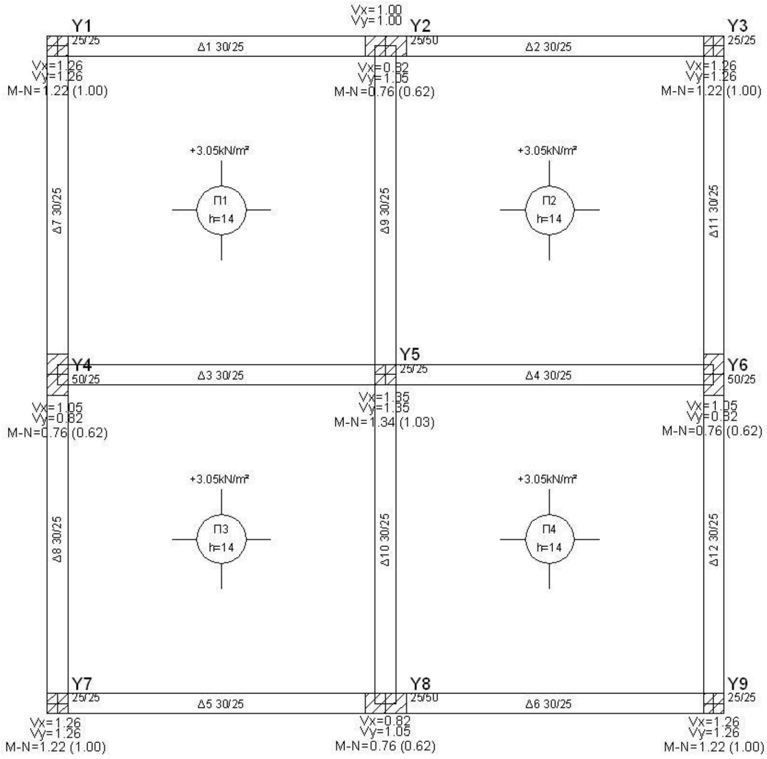

Figure 12: Structure 11

However, the redistribution of shear forces presents the same characteristics as for the structures 8 and 9 but it is less intense. A reduction by 4% and 18%, respectively, to stiffer elements and an increase by 18% and 35% to the less stiff elements is observed. This is another indication that the main role in the redistribution of shear forces belongs to the difference in stiffness between vertical elements while the position of the stronger and weaker elements is secondary but not insignificant. In structure 11 the central column, mainly, and the corner columns become insufficient in view of the M-N criterion.

Structure 12, in Figure 13, has two stiffer columns only in the x direction to which, under the fixed base assumption, corresponds 55% of the total base shear force. As a result its basic modal period is different in the two main axes (0,18sec in x direction and 0,23sec in y direction). This column arrangement causes, under SSI conditions, an 11% increase of total base shear force in the x direction, while in the y direction remains virtually unchanged. The redistribution of shear force, when SSI is considered, presents the same features as in the previous structures, but more intense. Specifically, the stiffer elements experience a reduction 11%, while the less stiff elements are called to withstand up to 51% more shear force.

Structure 13, in Figure 14, and structure 14, in Figure 15, follow a similar arrangement of vertical elements as in structure 11, with the stiffer columns in the perimeter, in both directions, which resist 50% of the total base shear force under fixed-base conditions. However, the spans between columns are not equal in both directions. The results of the analysis show that the redistribution of shear forces, when SSI is considered, is reaching up to 45%, at the weak central column which is connected to stiffer columns in both directions. The corner columns, connected to stiffer columns in one direction, are also strongly affected. Similarly, the central column of structure 13 and one corner column of structure 14 are strongly adversely affected as far as the M-N criterion is concerned, when SSI is considered. Thus, the proximity to stiffer elements seems to be another decisive factor to the redistribution of shear forces.

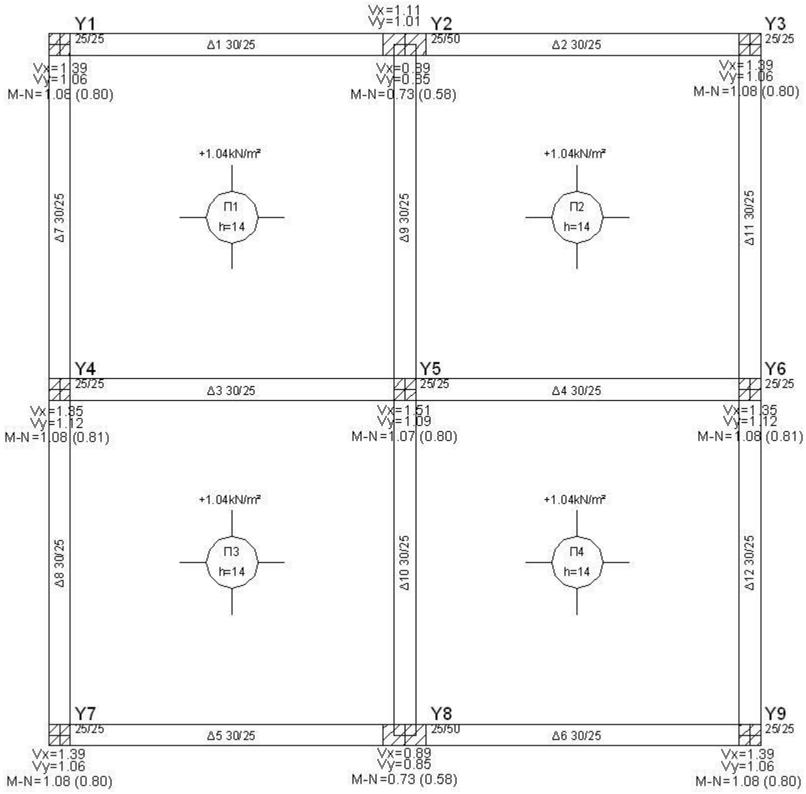

Figure 13: Structure 12

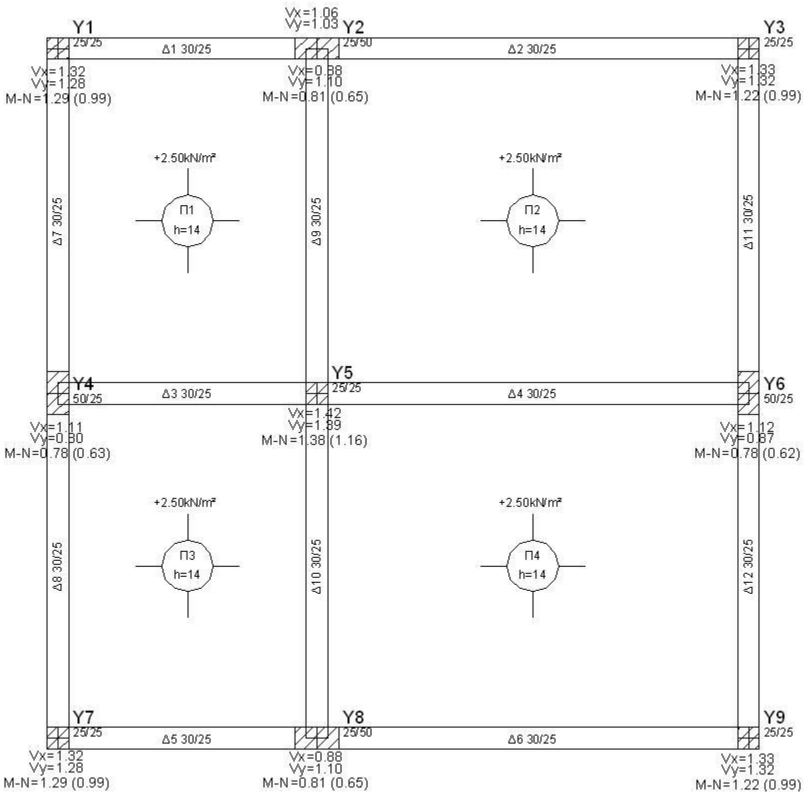

Figure 14: Structure 13

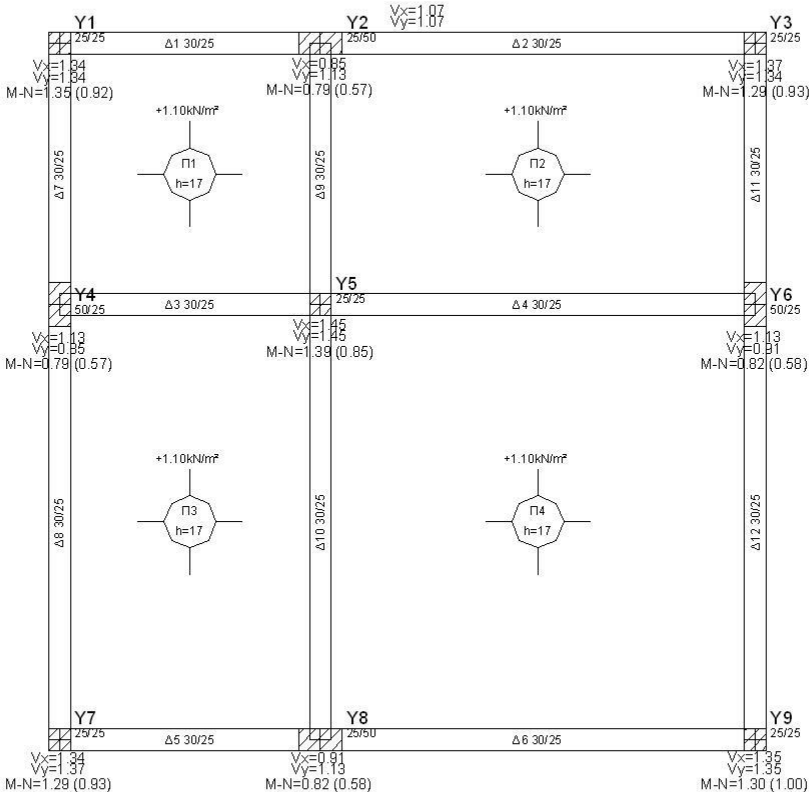

Figure 15: Structure 14

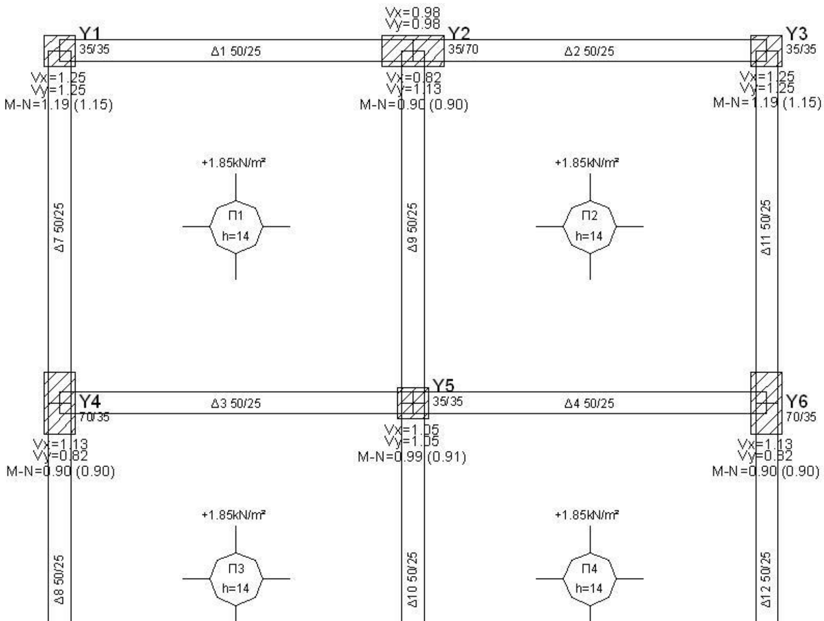

Figure 16: Structure 15

Finally, the layout of the ground flour of a three-storey structure 15 is shown in Figure 16. Like in structures 11, 13 and 14 the stiffer columns are located in the perimeter, in both directions, and they resist 55% of the total base shear force under fixed-base conditions. Under SSI conditions the total base shear force remains almost the same as that recorded with the fixed-base assumption (reduction by 2%). However, when SSI is considered, the stiffer columns are relieved by 18% in their strong direction while overloaded by 10% in their weak one. Now, under SSI conditions, the weak corner columns are overloaded by 25% increase in shear force, while, in contrast to previous examples, the central column experiences a smaller increase by only 5%. Curiously enough, this trend is reversed at the first floor where the central column becames more stressed and the ones at the corners are relieved. In any case all weak vertical elements at the corners are insufficient according to the M-N criterion.

A more detailed discussion on these matters and more such analyses can be found in Mantzaras [5].

4 CONCLUSIONS

The aim of this investigation is to quantify the role of soil-structure interaction, as compared to the usual code-specified fixed-base assumption, in the distribution of shear forces among the vertical elements of 3-D framed structures. To this end, simple concrete structures are designed according to pertinent code provisions and subjected to seismic excitations, consistent with design spectra. The soil-structure interaction is materialized through a set frequency independent springs, dashpots and lumped masses at the soil-foundation interface. The basic conclusions drawn from these analyses are as follows:

1. A substantial redistribution of shear forces among the various columns of a structure occurs when SSI is considered, in reference to the code specified fixed-base condition.
2. The basic factor affecting the redistribution of shear forces is the difference of stiffness among columns.
3. In general, when SSI is considered the stiffer vertical elements are relieved from the load they are called to carry under the assumption of fixed-base conditions. The opposite is true for the less stiff elements which, when SSI is considered, experience substantially higher shear loads. The numerical studies presented in this work have recorded increases under SSI conditions at levels above 50% to those calculated with the fixed base assumption.
4. The most affected of the less stiff elements are those directly connected to substantially stiffer elements or at least are in close proximity to them.

It should be noted that all the analyses presented in this work are linear elastic for the entire soil-foundation-structure system. They also pertain to structures that are supported on an elastic foundation system consisting of distinct footings interconnected via tie beams.

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