Comparison of Seismic Behavior of Multi-Storey R/C Buildings With and Without Internal Beams

I. A. Tegos
Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

V. P. Panoskaltsis
Department of Civil Engineering, Demokritos University of Thrace, Xanthi, Greece

S. D. Tegou & M. Papadopoulos
Department of Civil Engineering, Aristotle University of Thessaloniki, Greece

SUMMARY:
In the present study the seismic response of flat slab reinforced concrete structures is investigated. In particular, a multi-storey frame structure designed according to the current codes, Eurocode 2 and Eurocode 8, is used as a benchmark. The aforementioned frame structure is studied as a flat slab structure, after the removal of the internal beams. The vertical structural members, as well as the perimetric beams are preserved as in the original structure. The analytical investigation is performed by means of response-spectrum analysis, implemented within the FE commercial code SAP 2000. The conclusion of the study is that it is possible to construct reinforced concrete structures without beams, as long as the beams in the perimeter of the building are kept (i.e. there exists a minimum number of frames) and that most of the earthquake action (i.e. at least 90% of the seismic forces) is undertaken by suitably designed shear walls.

Keywords: Reinforced concrete structure, beam, slab, shear wall, core, shear punching

1. INTRODUCTION

It has been a frequent practice to design reinforced concrete structures without the use of beams, in particular in areas with low seismic activity. The important advantages of this approach versus the classical one (i.e. the one with beams), are the shorter construction time and therefore the lower cost involved. Moreover, there are significant and attractive architectural implications resulting from the lack of beams. Those are, more architectural flexibility within each floor as well as reduction of the overall multi-story building's height. It has been estimated that for a six-story building the height may be reduced by one floor.

Notwithstanding the aforementioned important advantages, many structural engineers hesitate following this approach, their major concern being the response of these structures to earthquakes (Erberik and Elnashai 2003). In particular, structural engineers were influenced after the major earthquakes in Central America in the 60's. It should be mentioned that two recent seismic codes, namely the Eurocode 2 (2003) and Eurocode 8-Part 1 (2003), permit the design of reinforced concrete structures without the use of beams in the perimeter, if some conditions are met.

In this paper and in order to address these questions the feasibility of designing reinforced concrete structures without the use of beams in the interior, in areas with high seismic activity is studied. It is well known that in this case a number of shear walls should be included in the structural system. An advantageous result of that is the reduction of displacements in the areas (finite joints) where the slabs are supported by the columns. A consequence of this, is the reduction of the internal forces. It should be mentioned that the connections of slabs and columns are the most sensitive areas of the structure because of the brittleness characteristics of the probable shearing punching mode of failure (Hueste and Wight, 1997; Megally and Ghali, 2000).

In order to accomplish this, a typical nine story reinforced concrete structure with a classic structural system was studied, see Fig. 1.1. Then, by removing the beams in the perimeter but keeping all the other structural elements, an alternative structure was formed from this, Fig. 1.2. Both representative structures, the “conventional” and the “new”, are analyzed in this work, against all actions dictated by the codes.

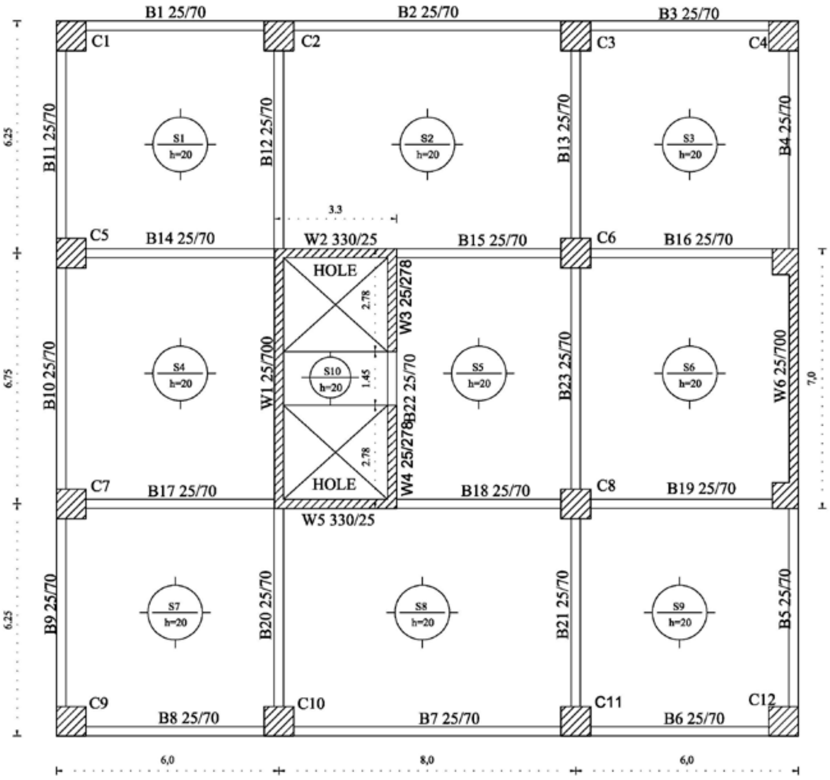

Figure 1.1. Plan view of a typical floor of the “conventional” building

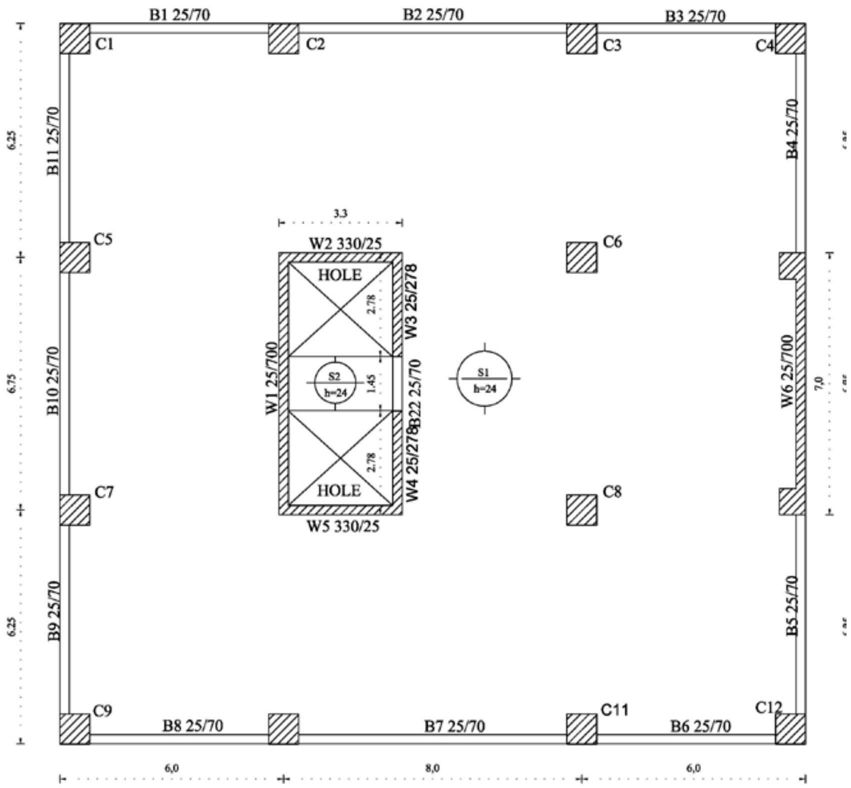

Figure 1.2. Plan view of a typical floor of the “new” building without internal beams

2. DESCRIPTION OF THE STRUCTURE

A reinforced concrete building consisting of a ground floor and eight additional stories is studied; the height of the ground floor is equal to 6.0m while each typical floor has a height h = 3.0m. The building's area of the ground plan is E=385.0 m^2, while its total height is H_{tot}=30m. It should be mentioned that there is no basement in the building and that for its foundation a slab-on-grade foundation has been chosen. The foundation's depth is equal to 2.0m and the slab's depth is equal to 1.0m. The soil's allowable stress was determined, by a suitable geotechnical study, to be σ_{all}=260 kN/m^2. The soil is in the group B according to the Greek Earthquake code (Ministry of Public Works 2000). It is noted that the corner periods of the spectrum used are 0.15s and 0.60s for ground type B, and this corresponds to a ground type between B and C according to Eurocode 8 (Eurocode 8- Part 1 2003). The building is located in an earthquake zone II (with peak ground acceleration α_g=0.24), i.e. it is in a high seismicity region. The concrete that was used for the building's construction is classified as C25/30 and the high ductility steel that was used as B500C.

A distinctive feature of the structural system of the structure under consideration is the existence of shear walls in the areas of the stairways and of the elevators shafts, as well as the existence of a very strong shear wall in the perimeter of the structure having length 7.0m (in the y direction of the ground plan).

The depth of the slab of a typical floor is determined from the reasonable choice to avoid checking the bending displacements. This check showed that a depth equal to 240mm is required.

3. MODELING, MATERIAL PROPERTIES AND ANALYSIS

The analysis of the structure was performed with the aid of the finite element method, which is clearly the most suitable method for a complex structure such as this one. For modeling of the slabs 3-node or 4-node shell elements were employed. These elements have 6 degrees of freedom per node and they, therefore, provide the possibilities of modeling a membrane type of behavior as well as a 3d plate bending behavior. In the areas in which a stress concentration is expected (i.e. in the support areas of the slabs as well as in the corners of shear walls and cores) the finite element mesh is refined, see Fig. 3.1.

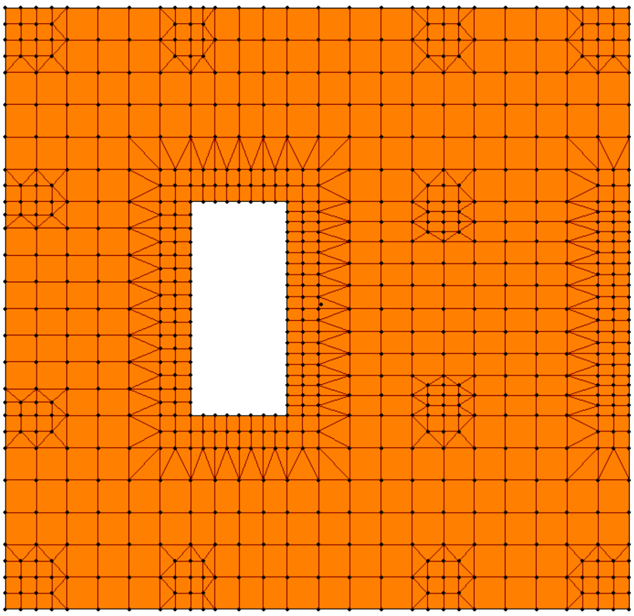

Figure 3.1. The finite element mesh of the concrete slab

It should be noted that the method of the equivalent frames (described in the codes), was not attempted for the analysis of slabs without beams, here. The reasons are the lack of a regular arrangement of the vertical structural elements, as well as because of certain deficiencies and weaknesses of this method, with respect to the vertical load decomposition along the x and y directions.

The cores were modeled as equivalent columns at the centers of the shear walls, and as rigid horizontal beams at the floor levels. The individual shear wall was modeled in the same way.

The modulus of elasticity for concrete was assumed to be E_{cm}=31GPa, corresponding to a C25/30 concrete. Element stiffnesses were reduced since we assume that the structural elements have entered into stage II (Eurocode 2-Part 1 2004). Under this assumption the stiffness of all elements, beams, columns and walls is taken equal to 1/2 of the stage I value. The torsional stiffness of the elements is assumed equal to 1/10 of the stage I value. The resulting 3d model is shown in Fig. 3.2 and is implemented into the finite element environment SAP2000 (Computer and Structures, 2007).

For the analysis of the structure under earthquake effects the linear modal response spectrum analysis was employed (Chopra 1995). This is the most general method, applicable to all types of structures covered by Eurocode 8-Part 1 (2003). According to this method, the total mass of each floor is assumed to be lumped at the floor's center of mass. The masses of columns and shear walls are included in a symmetric manner in the mass of each floor, i.e. in the mass of each floor we include half of the total mass of columns and shear walls extending from the upper to the lower of the floor under consideration. Moreover, it is assumed that the slabs behave as diaphragms, under the action of horizontal earthquake loads. This is a reasonable assumption because of the building's symmetry and the shape of the ground plan. The behavior factor is taken equal to 3.0 according to Eurocode's 8 provisions.

The masses are calculated from the vertical loads $\Sigma G_{k,j}+\psi_{E,i}Q_{k,i}$, where G and Q are the values of permanent and variable loads and ψ_{Ei} is the combination coefficient for the variable action i, which according to the code for reinforced concrete structures (Eurocode 0 2002; Eurocode 8- Part 1 2003), is considered to be 0.3, for dwellings, and offices. It should be mentioned here that the permanent loads G as well as the variable loads Q, act on the whole area of each slab.

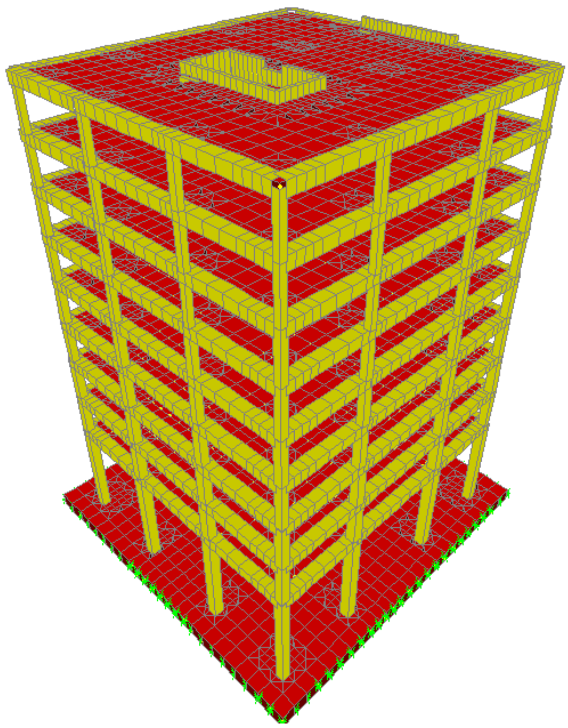

Figure 3.2. The 3d finite element model of the structure

It is important to emphasize that the analysis and dimensioning of the structure's foundation was achieved by studying the whole structure's interaction with the foundation's slab-on-grade. The foundation's slab is assumed to be supported on the ground via linearly elastic springs (Winkler foundation). The soil's modulus of resistance and therefore the springs' stiffness is assessed from the structure's settlements and is taken to be k_s=2500 kN/m^3. The foundation's slab was modeled with the finite element method; 2d shell elements were used, as in the floor slabs discussed earlier. The depth of the foundation slab was taken to be equal to 1.0m.

4. DIMENSIONING OF THE STRUCTURE

4.1. Slab's dimensioning against bending

Dimensioning of the slab of a typical floor under bending was performed for the following load combination 1.35G+1.50Q, and the adequacy of the resulting reinforcement was checked against the earthquake loading combination G+0.3Q±E.

4.1.1. Loading Combination 1.35+1.50Q

Using the computed bending moments M_{11} (i.e. in the direction x-x), Fig. 4.1(a), and M_{22} (i.e. in the direction y-y), Fig. 4.1(b), from our previous analysis, the dimensioning of the slabs was performed by using the strips shown in Fig. 4.2 and Fig. 4.3. The borders of the strips are defined by the middle of the distances between the vertical supports of the structure.

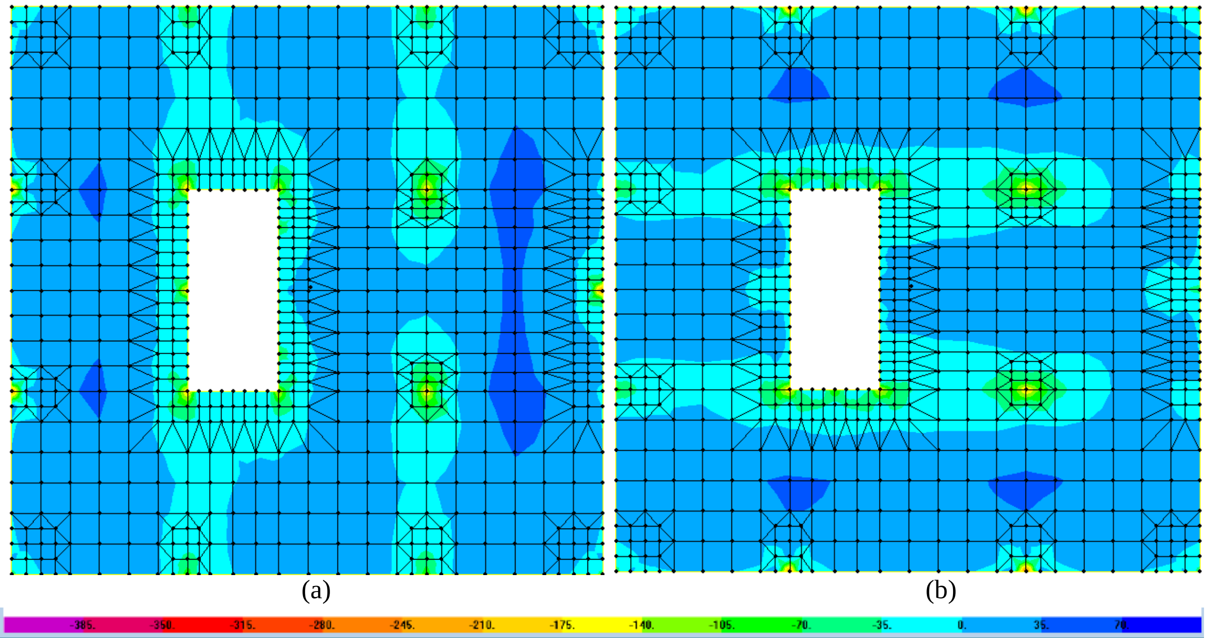

Figure 4.1. The bending moments (a) M_{11} (in the direction x-x) and M_{22} (in the direction y-y) of a typical slab

For reinforcement of the slabs, a reinforcement grid of D 10@200 is used, in both the upper and lower regions of the slab, extended in the whole floor plan, with additional reinforcement in the areas needed, according to the checks performed (see Fig. 4.4).

The fact that reinforcement was used in both the upper and lower regions of the slab, although it leads to more material used, it is nevertheless more economical in the end, because of the less labor required. This choice is clearly advantageous technically and as far as safety is concerned since the stress and strain fields resulting from temperature gradients and shrinkage effects are dealt with in a more efficient way.

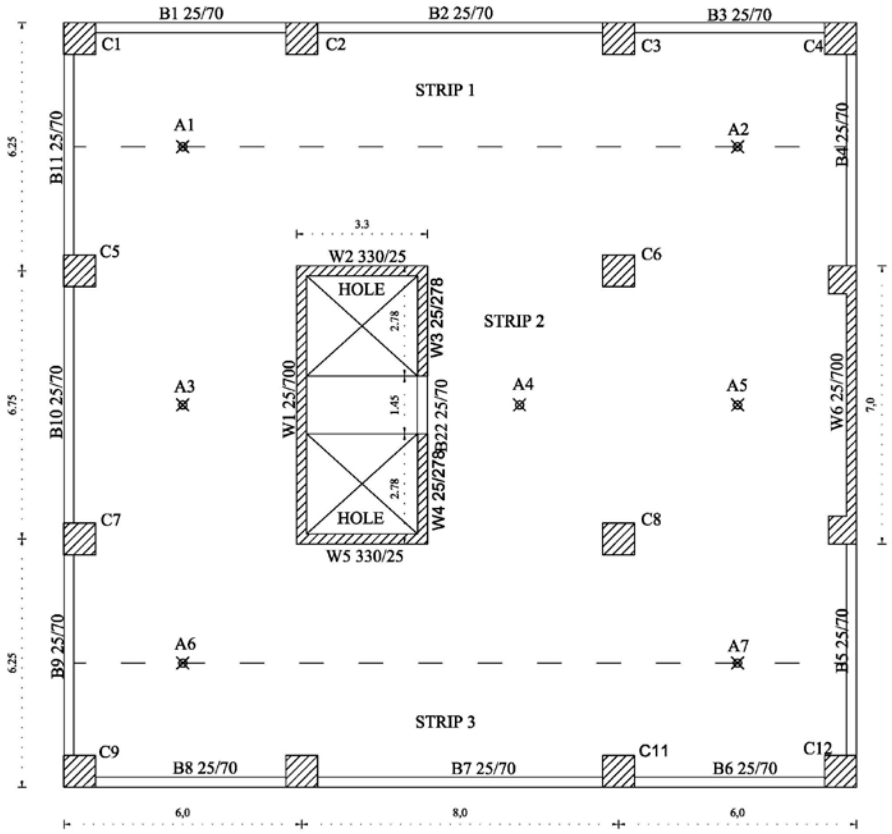

Figure 4.2. Strips used for the bending dimensioning of the slab in the x-x direction

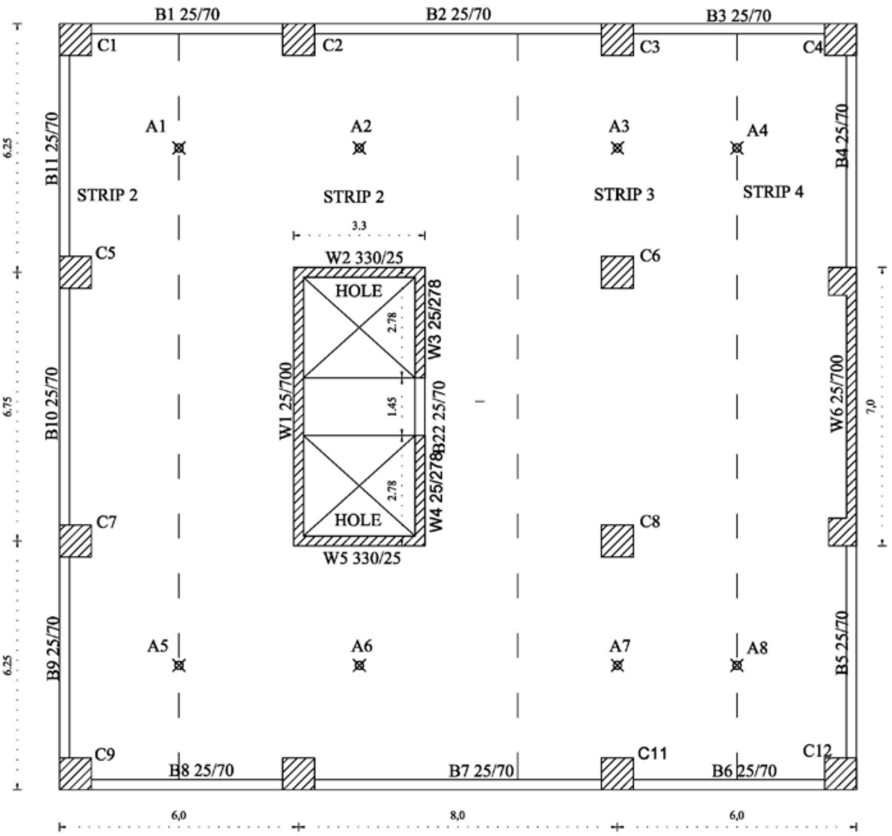

Figure 4.3. Strips used for the bending dimensioning of the slab in the y-y direction

4.1.2. Loading Combination G+0.3Q±E

The reinforcement resulted from the previous loading combination (i.e. 1.35G+1.50Q) is sufficient for the seismic action combination G+0.3Q±E considered herein, since in the supports the reinforcement mesh that was deposited is D10@200+ D10@100 (3.93 + 7.85 = 11.78 cm^2/m).

4.2. Slab's dimensioning against shear punching

The capacity design of the shear punching problem is addressed through the adoption of a reduced value for the behavior factor q. According to Eurocode 8- Part 1 (2003), the value of the behavior factor q is taken equal to 3 for systems of shear walls acting like cantilevers; this is how in our problem, the slabs without beams are acting. Nevertheless, in order to dimension the slabs against shear punching, we considered the behavior factor q to be q=3.0/1.4=2.14, where 1.4 is the well-established value for the overstrength coefficient. This way, we assumed a high value for overstress against shear punching, since the dimensioning of the slabs against bending took place with q = 3.0.

The punching shear forces, along the critical perimeters, as these are defined in Eurocode 2 (2004) were computed with the aid of the finite element program SAP2000 (Computers and Structures, 2007), and by using the option "assign groups". Specifically, the groups of joints defining the critical perimeter were assigned and with the aid of the finite element program the punching shear forces were computed by an integration of the stresses. With this procedure the punching shear forces were computed at the connections of the slabs to the columns and to the shear walls corners. This was accomplished for both loading combinations, i.e. for the loading combination 1.35G+1.50Q as well as for the seismic loading combination G+0.3Q±E. The control perimeter u is calculated according to paragraph 6.4 of the Eurocode 2-Part 1 (2004).

From our analysis it is concluded that in all slab-column joints, apart from the corresponding ones of the 8th floor, the acting punching shear force v_{Ed} is smaller than the slab's strength in punching, when there is no punching reinforcement v_{Rdc} ($v_{sd} < v_{Rdc}$). Table 2 summarizes the aforementioned check implemented for the 8th floor. Therefore, at least in theory, there is a need for punching reinforcement only at this floor. Nevertheless, the same reinforcement is placed at all slabs for safety reasons. In Fig. 4.4, the layout of a typical floor with the bending and punching shear reinforcements is depicted. Fig. 4.5 illustrates the punching shear reinforcement detailing at the connections of the slab to the columns and to the shear walls' corners.

Similar procedure is followed for the bending and punching shear design of the foundation's slab. In Fig. 4.6, the layout of foundation's slab with the bending and punching shear reinforcements is depicted.

5. CONCLUSIONS

In this work, the effects resulting from the elimination of the interior beams of a multi-story building of a large ground plan were studied. The emphasis was placed on punching shear concerns, resulting from the direct support of slabs on columns and shear walls. The requirements of both Eurocode 2-Part 1 and Eurocode 8- Part 1, were also followed. The structure was treated as a 3d configuration and the analysis was performed within a finite element framework. In summary the main conclusions are the following:

1) The shearing punching of the slabs, at the points of direct support on columns is not critical. This becomes critical at the corners of the shear walls cores.

2) Regarding shear punching, it is seen that the earthquake action is a little more critical than static loads.

3) The slab's depth was determined from maximum bending deflection criteria. It was seen that the so

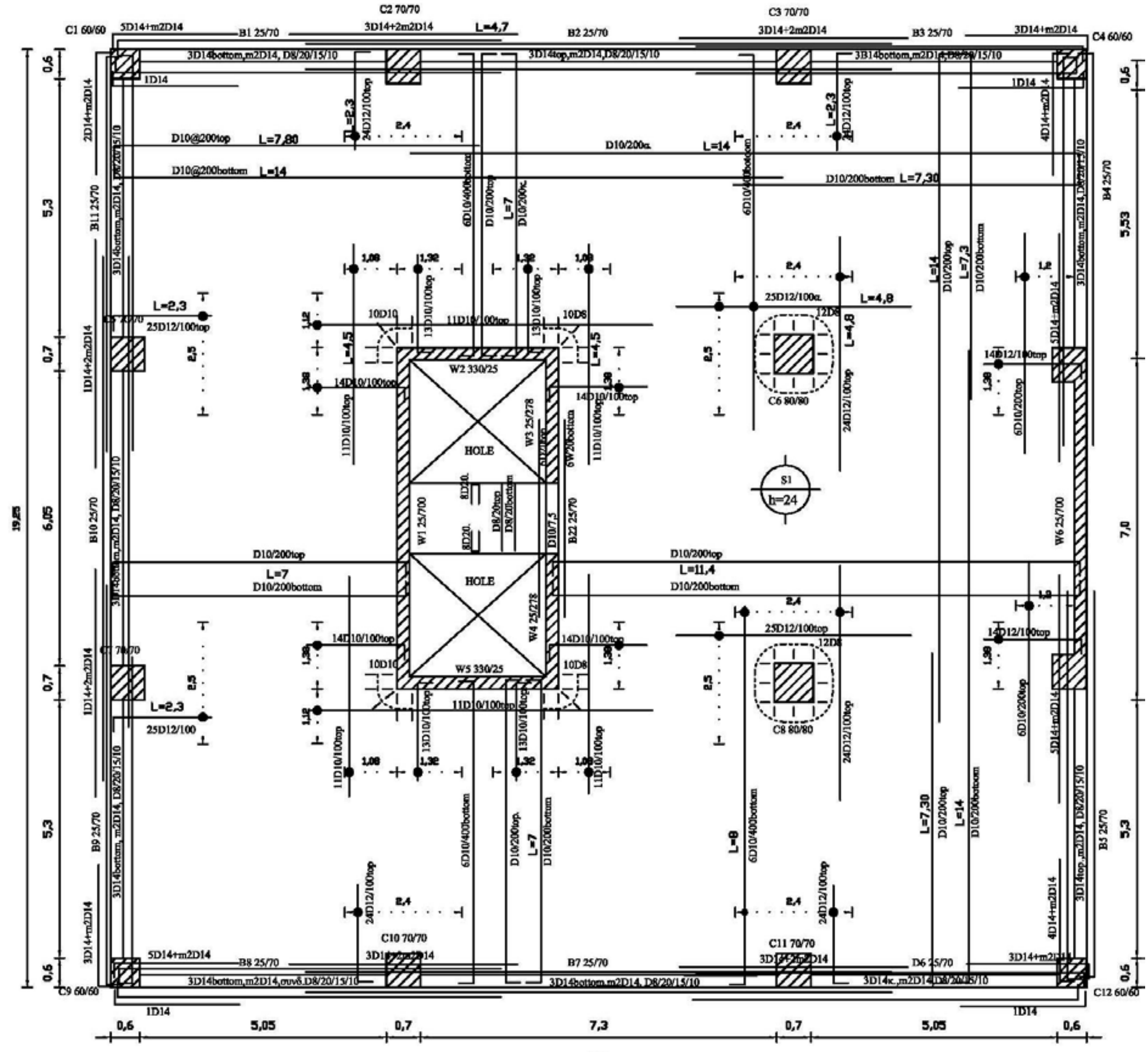

Figure 4.4. Reinforcement layout of the slab of a typical floor

Table 4.1. Punching shear check for the members of the 8th floor

Combination	Member of 8th floor	control perimeter (m)	Design value of the applied shear V_{Ed} (kN)	Punching shear stress v_{ed} (MN/m^2)	maximum punching shear resistance v_{Rdmax} (MN/m^2)	punching shear resistance without reinforcement $v_{Rd,c}$ (MN/m^2)
1.35G+1.50Q	C6	4.11	561.62	0.683	4.5	0.64
	C8	4.11	566.16	0.689	4.5	0.64
	W1-W2	1.23	246.71	1.003	4.5	0.59
	W2-W3	1.23	246.46	1.002	4.5	0.59
	W4-W5	1.23	197.1	0.801	4.5	0.59
	W5-W1	1.23	198.75	0.808	4.5	0.59
G+0.30Q±E	C6	4.11	460.21	0.56	4.5	0.64
	C8	4.11	463.3	0.564	4.5	0.64
	W1-W2	1.23	261.89	1.065	4.5	0.59
	W2-W3	1.23	222.48	0.904	4.5	0.59
	W4-W5	1.23	223.03	0.907	4.5	0.59
	W5-W1	1.23	261.94	1.065	4.5	0.59

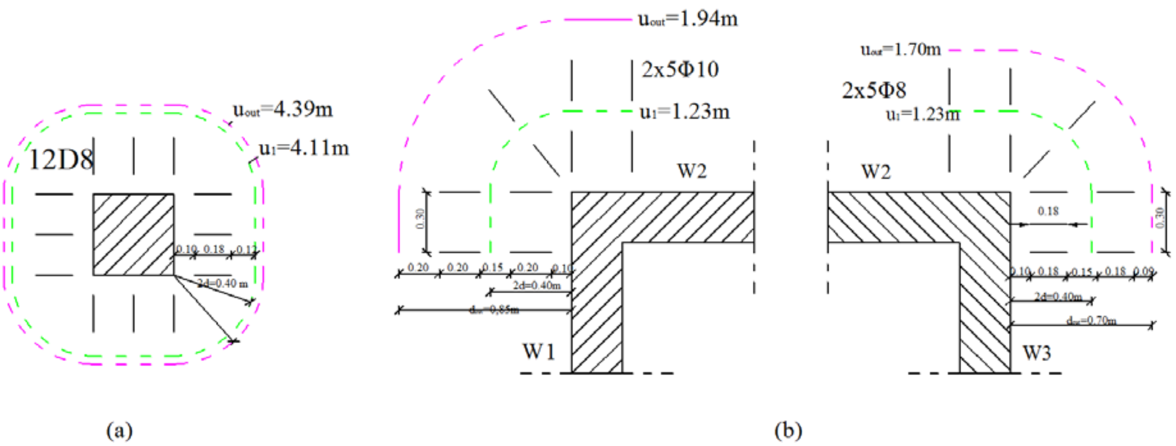

Figure 4.5. Punching shear reinforcement detailing at the connections of the slabs: (a) to the columns and (b) to the shear walls corners

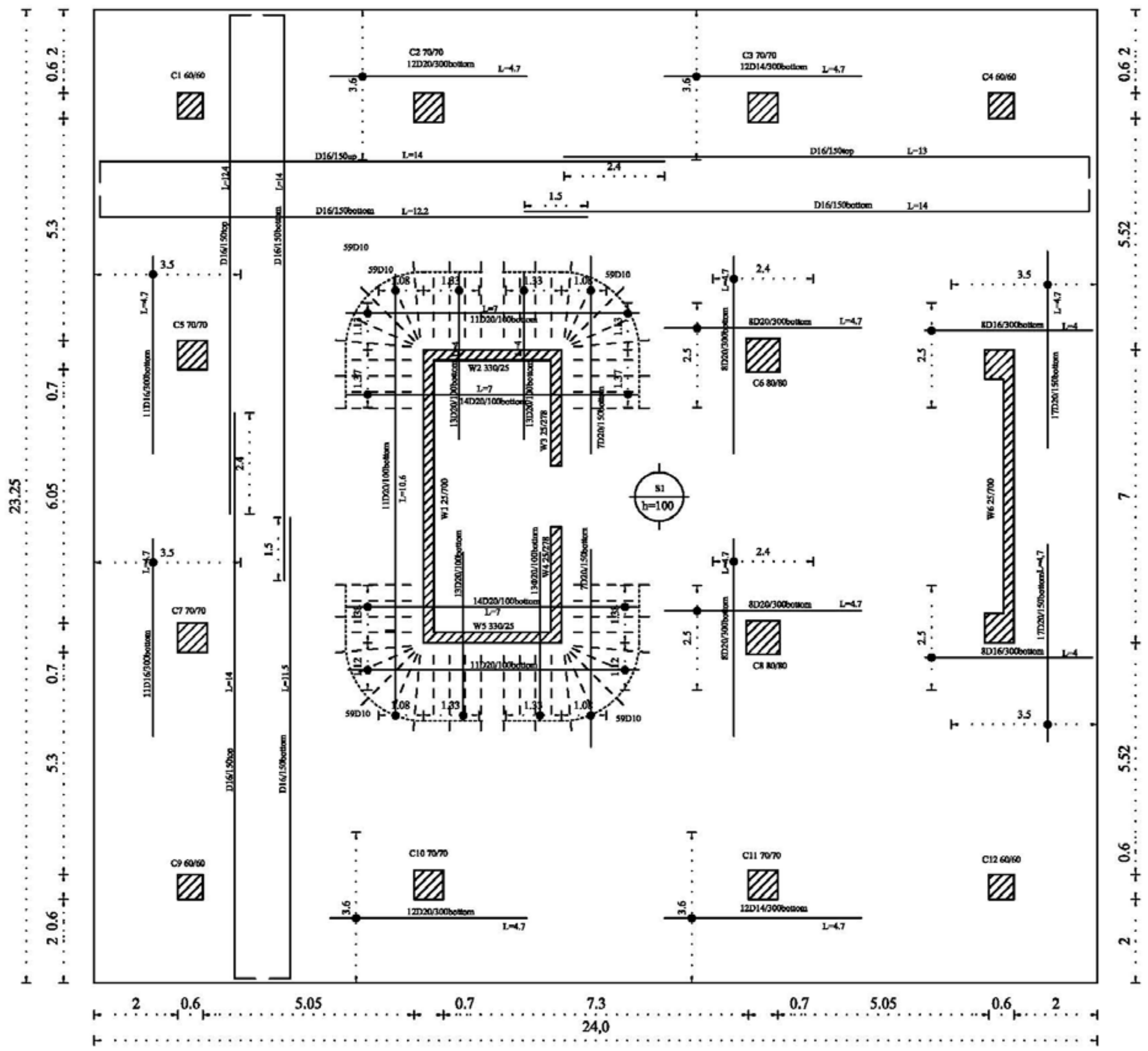

Figure 4.6. Reinforcement layout of the slab-on grade foundation

determined depth is marginally adequate against seismic loadings. Therefore, increase of elements' dimensions locally is not needed. It should be noted that the satisfaction of depth requirements was established under capacity design considerations, i.e. the performance coefficient q was reduced from 3.0 to 2.14, as it was explained earlier.

4) If in some regions the depth is less than the required, beams could be locally added (usually this happens when two vertical elements are very close to each other). Otherwise, this problem can be solved by adding more shear walls in the (3d) structural system. This results to smaller floor displacements and consequently to smaller shear punching stresses.

5) It should be remembered that stress spikes resulting from the shear punching will be "captivated" from the neighboring region; this is similar to the familiar situation of stress spikes existing in the corners of holes in slabs.

6) It should be noted that the reinforcement mesh, which is put in the upper and lower surfaces of the slabs, although it may seem at a first glance to increase the structure's cost, it is advantageous regarding the structure's easiness of construction. Furthermore, the reinforcement mesh is acting against cracking, in particular in structures in which the length of the floor plan is greater than 30m.

REFERENCES

Erberik, M. A. and Elnashai, A. S. (2003). Seismic vulnerability of flat-slab structures. Mid-America Earthquake Center CD Release 03-06.

CEN [Comité Européen de Normalisation] (2002), Eurocode 0: Bases of structural analysis.

CEN [Comité Européen de Normalisation] (2004) Eurocode 2: Design of concrete structures-Part 1: General rules and rules for buildings.

CEN [Comité Européen de Normalisation] (2003), Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings.

Computers and Structures Inc. (2007) SAP 2000 Nonlinear Version 11.0.3, User's Reference Manual, California.

Chopra A.K. (1995) Dynamics of Structures, Theory and Applications to Earthquake Engineering, Prentice Hall Inc.

Hueste, M.B.D. and Wight, J.K., (1997), Evaluation of a Four-Story Reinforced Concrete Building Damaged During the Northridge Earthquake. *Earthquake Spectra*. **13:3**, 387-414.

Megally, S. and Ghali, A. (1994). Design Considerations for Slab-column Connections in Seismic Zones. *ACI Structural Journal*. **91:3**, 303-314.

Ministry of public works of Greece (2000), Greek seismic code-EAK2000, Athens, (Amended June 2003).