Antiseismic study of 15-storey building without beams, of large layout and large spans: Inspection of seismic movements and raft foundation

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Abstract

The purpose of this research is to prove the feasibility of constructing buildings without beams at locations with high seismicity. The finite elements model used, involves big internal spans between the supports, central shear walls core system and no internal columns, all of which lead to the application of prestressing tendons. After the application of the gravity loads and the transaction of dynamic spectral analysis, the study focuses on restricting the harmful horizontal displacements caused by seismic stresses. To achieve that, different ways of simulation of coupling beams are explored, in order to connect the shear walls to each other. Another concern of the study is to preserve the vertical displacements of the slabs inside acceptable limits, by adjusting the position and geometry of the tendons. Subsequently, the research deals with the treatment of the bending stress requirements of the components, as well as the slab punching shear problems. The last part of the study examines the possibility of activation of the entire body of the raft foundation in order to receive the vertical building loads. The results that emerge will, hopefully, dispel the doubts that accompany this sort of constructions at high seismicity areas.

Key words – Grid slab; prestress; high seismicity; coupling beam; seismic movement; raft foundation.

Introduction

It is well known that the existence of beams in a construction contributes to the satisfaction of safety and functionality criteria. Apart from these two basic criteria, however, there are also the criteria of aesthetics, constructability and economy which plead on behalf of constructions without beams due to the lack of need to use suspended ceilings and for the interior walls to follow the beam lines. Moreover, due to the absence of beams it is possible for every six floors to gain the height of one floor. Another advantage of such type of constructions is the drastic reduction of the required construction time.

In spite of all these important advantages, there are some disadvantages that occur caused, mainly, by the earthquake factor. These are the danger of the failure of the slabs due to the punching shear as well as the damaging seismic movements which fuel the P-Δ effects, especially in tall buildings.

So the study examines a 15-storey building without beams located at a relatively high seismicity zone with design ground acceleration value \( a_g = 0.24 \cdot g \).
The characteristics of the construction are:

- 40x40m² floor layout (Fig.1)
- Circular cross section columns on the perimeter (1m diameter)
- Typical floor height = 3.2m
- Shear wall dimensions: 40x300cm²

The floor slab is the greater part of the building’s weight, therefore it is imperative to achieve the best economic solution. So the slab cross section is kept solid only at the areas which are critical against punching shear, specifically on the slab perimeter and around the central shear wall system. At the rest of the areas a grid slab type is selected so that the structure self weight remains at low levels (Fig.2).

The basic parameters concerning the study are:

- The extent of the slab spans
- Shear walls presence
- Dimensions and geometry of the floor plan
- Use of prestress tendons
- Shear wall coupling beams
- Placement of peripheral parapet
- Basement presence

The basement provides the structure with several advantages such as the beam function of the perimeter concrete walls, the resistance to the lateral soil pressure and the significant reduction of the base bending moments (Tegos 2009).

The placing of the prestressing tendons forms an important part of the current study. The big slab spans (16m) lead to them being essential to the construction. The tendons have the ability to prevent the punching shear danger through the appearance of deviation forces which also contribute to the containment of the vertical slab deflections and the quantity of the conventional steel reinforcement required. A parabolic tendon geometry is preferred so that it follows the bending moment diagram form caused by the external loads. In that way the positive effects of the prestress are maximized. Regarding the arrangement of the tendons on the slab floor, a dense arrangement of tendons is used.
for the stripes passing around and close to the slab vertical supports. At the rest of the slab a sparser arrangement of tendons is used. In this way an effective and, simultaneously, economical solution of prestress use is achieved (Rombach 2010).

The relatively big height of the building, at least by the standards in Greece, leads to high fundamental period values of the order of 1.6 to 1.7 sec. However, the rare occurrence of big depth earthquakes in Greece which affect the constructions with high fundamental periods results in low inertial forces to the construction.

Extended investigation is conducted on the horizontal seismic movements which affect the construction. At this case the importance of the coupling of the shear walls through beams on the reduction of the movements is examined. Special care to the type of reinforcement used for these beams must be taken in order to ensure that brittle failure will not happen. Besides, further increase of the bending and rotational stiffness of the construction is achieved by placing parapets on the perimeter of every floor.

**Methods**

For the linear analysis of the construction finite element method was used, executed on SAP2000 software. The slab and the basement perimeter walls are modeled by using 3-noded and 4-noded plate bending and membrane finite elements. A densification of the slab elements is applied at the slab areas where high stress concentration is expected, specifically close to the slab support areas. Every node has 3 degrees of freedom (dof). The rest of the load bearing elements are simulated with beam objects. The shear walls are modeled with equivalent columns at the centre of gravity of theirs and rigid beams at every floor level. To simulate the wall coupling beams more than one ways were used to be analyzed later.

The modulus of the concrete obtained equal to $E_{cm} = 35$ GPa, corresponding to category C40/50. The stiffness of the elements is reduced assuming stage II (cracked cross section). Thus, the bending stiffness obtained for the columns and the shear walls is equal to half of that of stage I, without taking into account the contribution of the reinforcement (geometric cross-section stiffness) and the coupling beams stiffness also equal to half the stiffness for stage I (Eurocode 2). As regards to the slabs, the serviceability limit state checks, due to the existence of prestress concrete, were executed in stage I whereas the ultimate limit state checks were executed assuming cracked cross section (stage II). The rotational stiffness of all the members is taken equal to the $1/10$ of that of stage I.

For the calculation of the seismic response of the building dynamic spectral method was used, which applies, without restrictions, in all construction situations covered by Eurocode 8. The total mass of each floor is assumed to be concentrated at the layout geometric center and consists of half the height of the overlying and underlying vertical components of each floor as well as the slab gravity loads. Moreover, the slab diaphragm behavior is taken into account for horizontal seismic loadings due to the conditions of structural regularity criteria being met. That was necessary in order to reduce the amplitude of time-consuming calculations, due to the presence of the tendons and the numerous nodes of the slab elements.

**Prestressing tendons simulation**

Initially, the vertical deflection values of the slab, before the tendons are applied, are compared with the span over 250 value ($L/250$). That limit is surpassed on all but one point of those examined so the application of prestress is necessary. Two types of tendons are tested. A conventional one and a flat tendon type with flat anchorages especially adapted for the post-tensioning of thin elements such as the concrete slabs.
The limit of L/250 is not surpassed after the application of the tendons.

In order to determine the appropriate tendon arrangement on the layout and the distance between them, the normal stress values of the slab are compared to the characteristic axial tensile strength of the concrete ($f_{ck} = 2.5\text{MPa}$). The comparisons are carried out for the $g+g+q+P$ load combination and uncracked slab cross section (stage I). The comparison resulted, both for the conventional and the flat
tendons, in exceedance of the limit only at restricted areas of the slab, a fact that can be accepted because of the slab ability to reshuffle the stresses across its body.

The flat tendons and their arrangement is, finally, chosen as the solution used for the calculation of the required reinforcement for the slab.

**Coupling beams simulation**

The high reinforcement requirements of this type of beams lead to investigation of several cases of simulation. The first one is with simple frame section elements to which are given the dimensions of the beam (40x100cm).

The second simulation attempt concerns the introduction of the diagonal bars type of coupling beams reinforcement (Fig.6).

![Figure 6. Diagonal bars reinforcement drawing](image)

For this type of reinforcement, link elements are used. The value of axial stiffness of the link elements is given by formula 1.

\[
K = 1.2 \cdot 1.25 \cdot \frac{E_s}{l} \cdot A_s
\]

(1)

The 1.2 and 1.25 constants are experimentally determined and introduce the positive effect of the behavior of the diagonal bars in tension and compression situation respectively.

The rotational degrees of freedom R1, R2 and R3 are fixed (no rotation allowed) because of the anchoring of the bars inside the wall or the slab, depending on the boundary conditions of the beam.

The third type of reinforcement is that of “roboidal” shape (Fig. 7). That type of reinforcement emphasizes on a more ductile behavior of the beams. On the other hand it requires big beam height and, therefore, it was applied only on the ground floor roof where there is 4.5m storey height.
Raft foundation simulation

The raft foundation slab is mounted onto the ground and is loaded underneath by the solid ground stresses. It contributes to the uniform apportion of the gravity loads to the entire raft foundation body, transferred to it by the vertical load bearing elements of the construction. The current foundation slab has a 1.25m thickness and solid cross sectional area. The floor layout is extended at each side of the slab with 4m cantilevers. Four flat prestress tendons per meter of slab are installed in each direction.

To simulate the behavior of the soil, springs with stiffness $K_s = 20000$ kN/m³ are introduced and attached on the downside of the area slab objects in order to simulate the soil behavior.

The tendon geometry is divided into two types depending on the change between positive and negative bending moments at the middle of every slab span.

Results

Horizontal seismic deflections inspection

The model is examined for six different cases:

1. No coupling beams (zero stiffness)
2. Coupling with frame section elements
3. Coupling with link elements
4. Coupling with “roboidal” type of reinforcement on ground floor roof level
5. Coupling with link elements and addition of parapet (40x100cm) on the perimeter of every storey level
6. Coupling with link elements and gradual reduction of the columns diameter

On figure 9 the resulting horizontal deflection values in x axis are presented. The represented values are a result of the analysis values times the behavior factor \( q=3 \).

![Linear anelastic deflections - x axis](image)

**Figure 9.** Horizontal seismic deflections diagram

Similar results are observed also in y axis. The main emerging conclusions are:

- The use of shear wall coupling beams causes the reduction of the horizontal seismic movements
- The addition of parapets on the perimeter of every storey causes further reduction on the horizontal movement values, especially at the higher floors.
- The case of reduction of the columns diameter has no significant impact, positive or negative, on the movement values.

**Structural design**

**Slabs - Bending**

In order to simplify the calculations it is needed to apply the reduction of the prestressing steel into conventional steel through the factor \( \lambda \) taking into account the yield stress and the effective depth of both types of steel .

\[
\lambda = \frac{1670}{500} \cdot \frac{d_{\text{end}}}{0.46} = 7.26 \cdot d_{\text{end}} \quad [d_{\text{end}}]=[m] \quad (2)
\]
The loading combinations for which the construction is examined are:

- Gravity loads combination \( \gamma_G G + \gamma_Q Q + \gamma_P P' \)
- Earthquake loads combination \( G + 0.3Q \pm E + P' \)

The final reinforcement selected includes the installation of reinforcing mesh Ø10/150 both in the upper and lower thin solid layer of the slab. Additional reinforcing bars are required, according to the calculations, only at the support areas and the solid areas at the slab perimeter.

**Slabs – Punching shear**

The critical areas against punching shear are the areas close to the support of the slabs. Therefore the perimeters of the columns and the corners of the shear walls are examined. The calculations resulted in no need of punching shear reinforcement. That can be credited to the fact that the tendons increase significantly the longitudinal reinforcement ratio and that the slab has increased effective depth. The result of the punching shear design strengthens the argumentation in favor of the economy of such a construction.

**Columns**

For the columns on the construction perimeter a longitudinal reinforcement of 18Ø25 is chosen and a double spiral 2Ø10/100 as confinement reinforcement.

**Central shear wall system**

Because of the absence of shear walls on the perimeter of the construction and, therefore, of much needed additional stiffness, significant requirements for reinforcement inside the confined boundary elements of the walls are created. Therefore an increase of the every wall width from 0.4m to 0.6m is decided. The calculations resulted in the placing of 68Ø20 reinforcing bars in every confined boundary area and Ø10/100 confinement reinforcement.

![Figure 10. Spots for investigation of vertical slab deflections](image)
Raft foundation

Vertical deflections

The target is to achieve the transfer of a part of the tension from the vertical elements areas to the, relatively, underused span areas through the installation of the prestressing tendons. That forces into action the whole body of the foundation slab against as gravity loads carrier. An investigation of the vertical deflections reveals the expected results. These are the reduction of the values at the vertical elements areas of the raft foundation and the increase of the values at the span areas.

Table 2. Vertical deflection values with and without the influence of prestressing

<table>
<thead>
<tr>
<th></th>
<th>( u_x ) (m)</th>
<th>( u_x ) (m)</th>
<th></th>
<th>( u_x ) (m)</th>
<th>( u_x ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Point</td>
<td>G+Q</td>
<td>G+Q+P</td>
<td>Point</td>
<td>G+Q</td>
<td>G+Q+P</td>
</tr>
<tr>
<td>B1</td>
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<td>-0.0096</td>
<td>B11</td>
<td>-0.0093</td>
<td>-0.0091</td>
</tr>
<tr>
<td>B2</td>
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<td>-0.0099</td>
<td>B12</td>
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</tr>
<tr>
<td>B3</td>
<td>-0.0073</td>
<td>-0.0051</td>
<td>B13</td>
<td>-0.0061</td>
<td>-0.0073</td>
</tr>
<tr>
<td>B4</td>
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<tr>
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<td>-0.0090</td>
<td>B16</td>
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</tr>
<tr>
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<td>-0.0066</td>
<td>B17</td>
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</tr>
<tr>
<td>B10</td>
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<td>-0.0078</td>
<td>B20</td>
<td>-0.0202</td>
<td>-0.0182</td>
</tr>
</tbody>
</table>

That can also be observed on the diagram of the ground stress towards the raft foundation slab along a tendons length (Fig 11).

Regarding the slab bending analysis, the tendons area covers the entire bending reinforcement requirement. Only a minimum conventional reinforcement quantity has to be used in order to prevent thermal deformations of the slab.

Figure 11. Ground stresses along the raft foundation slab
Conclusions

In this paper is examined for a building without beams, of large floor layout, big height for the Greek standards, with large spans and, consequently, sparse columns placed only on the perimeter of the building, with shear walls, for high seismicity, the impact of:

i. prestressing
ii. vertical component of seismic action
iii. presence of perimeter parapet
iv. horizontal seismic movements
v. wall coupling beams
vi. gradual reduction of column diameter
vii. basement presence
viii. general raft foundation

The main conclusions of the parametric investigation may be formulated as follows:

1. Prestress is a necessary choice because of the long spans of the structure and the long distance between the vertical components. The arrangement of the prestressing tendons follows specific rules in order to obtain the maximum possible performance, based on the criteria of economy, efficiency and punching shear prevention.
2. The absence of beams has negative and positive effects. In the case of the negative ones, they are mitigated by the presence of prestress, thanks to which a reduction of the slab thickness is achieved, as well as a reduction of the bending deflections and, as regards to the foundation, activation of the entire slab body in order the gravity loads to be carried.
3. The influence of the vertical seismic component is not a critical design factor.
4. The horizontal movements are significantly improved by adding perimeter parapets and due to the presence of coupling beams between the shear walls.
5. The escalation in height of the diameter of the small number of perimeter columns showed no significant deterioration in the horizontal displacements.

References


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