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OBSERVED BEHAVIOUR OF AN INDUSTRIAL COMPLEX WITH PRE-CAST R/C STRUCTURAL MEMBERS SUBJECTED TO THE ATHENS 1999 EARTHQUAKE - NUMERICAL SIMULATION OF ITS RESPONSE

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Abstract. *This paper studies the dynamic and earthquake behavior of an industrial complex that is located in Aharnai-Athens and was subjected to the strong ground motion of the Athens 1999 Earthquake. This industrial complex is mainly composed of a large industrial hall and an adjacent office building. The columns, beams, roof girders and roof trusses of the industrial hall are all pre-cast reinforced / pre-stress concrete structural elements. This pre-cast structural system is complemented with shear walls in both horizontal directions in the perimeter of the industrial hall which were cast in place and connected to the pre-cast structural columns. All the pre-cast beams are simply supported and connected to the pre-cast columns through metal anchor bars and/or extended ties. The foundation is also formed by pre-cast pocket elements and ground beams which are interconnected between with cast in-place parts. The office building is a three-storey structure with a basement that is separated from the industrial hall with a construction joint. The foundation and the columns of this building are cast in place whereas part of the beams and all the slabs are pre-cast and are joined with the columns through metal extended ties and cast in-place concrete. This industrial complex is located at the epicentral area of the Athens 1999 earthquake which caused considerable destruction to industrial and residential buildings located in the vicinity of the industrial complex that is examined here. The structural damage observed in this industrial complex was the dislocation of a heavy pre-cast girder at the façade of the office building as well as cracking of the horizontal beams at the locations where they joined the columns. On the contrary, no dislocation of the pre-cast girders could be seen at the industrial hall which performed rather satisfactorily with minor signs of distress. These two distinct structural systems were numerically simulated in an effort to study their dynamic and earthquake response and correlate it with the observed performance. The present investigation demonstrated that the*

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use of pre-cast reinforced concrete members, when properly designed and detailed, can be employed with confidence in seismic regions.

1 INTRODUCTION

This paper studies the behaviour of an industrial complex, formed by an industrial hall and an office building. They were partly constructed employing reinforced/pre-stressed concrete pre-cast structural members. This type of structures are characterized by relatively large spans and flexible connections between the vertical and the horizontal structural elements (e.g. between columns and girders). Their main advantage of pre-cast construction is the relatively short construction time when compared with traditional cast in-situ reinforced/pre-stressed concrete structures. However, such pre-cast systems exhibited in the past unsatisfactory performance when subjected to intense seismic excitations. This unsatisfactory performance is mainly due to the capacity of the flexible connections between the vertical and the horizontal pre-cast structural elements, as depicted in figures 1a to 1d; that is these connections fail to withstand the displacement and strength demands imposed on them by strong seismic actions

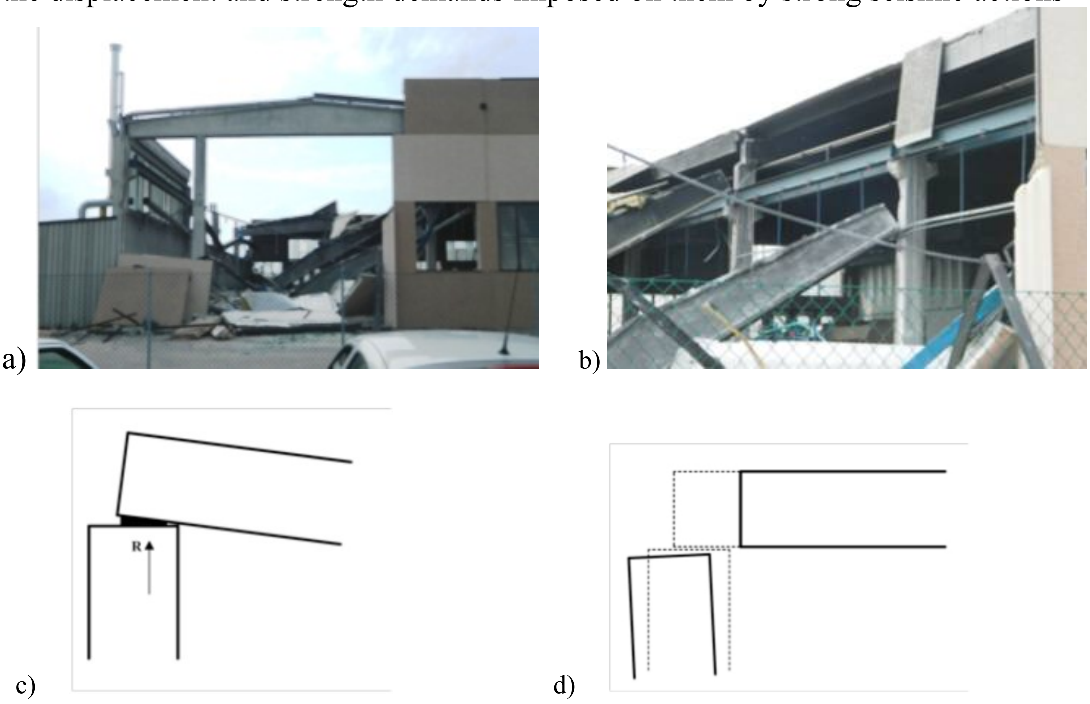


Figure 1. Unsatisfatcory performance of pre-cast industrial buildings due the displacement and strength limitations of the flexible connections between the vertical and the horizontal structural elements.

In the following sections the dynamic and earthquake behavior of an industrial complex is presented that is located in Aharnai-Athens and was subjected to the strong ground motion of the Athens 1999 Earthquake. The industrial hall is 77,24m x 24,75m in plan with 7,47m height from ground level to the level of the horizontal beams that support the roof and a height of 6,15m from the foundation to the ground floor that forms the space of the basement. The columns, beams, roof girders and roof trusses of this industrial hall are all pre-cast reinforced / pre-stress concrete structural elements. This structural system is complemented with shear walls in both horizontal directions in the perimeter of the industrial hall which were cast in-place and connected to the pre-cast structural columns. All the pre-cast beams are simply supported and connected to the pre-cast columns through metal anchor bars and/or extended ties. Its foundation is also formed by pre-cast pocket elements and ground beams that are interconnected with cast in-place parts. The office building is a three story structure with a basement. It is separated from the industrial hall with a construction joint. The foundation and the columns of this office building are cast in place whereas part of the beams and all the slabs are pre-cast and are joined with the columns through metal extended ties and cast in-place concrete. This industrial complex is located at the epicentral area of the Athens 1999 earthquake which caused considerable destruction to other industrial buildings located in the vicinity (see figure 2 and reference [1]) as well as to many residential buildings.

Figure 2. Partial or total collapse of industrial buildings during the Athens, Greece 1999 Earthquake

This industrial was under construction during the Athens, Greece Earthquake, as can be seen from figure 3 taken a few days after this seismic sequence that occurred on the 7th of September 1999. The ground acceleration that was recorded near the center of Athens by an instrument located 16km to the South of the epicentral region (ATH-3, Odos Pireos, with the yellow pin and the code name ΚΕΔΕ in Figure 3) had peak values equal to 2586mm/sec2 in the longitudinal horizontal direction, 2972mm/sec2 in the transverse horizontal direction and 1537mm/sec2 in the vertical direction. The epicenter of the 1999 earthquake is also indicated in figure 3 together with the exact location of the studied building complex (yellow pin with the code name ALUMIL).

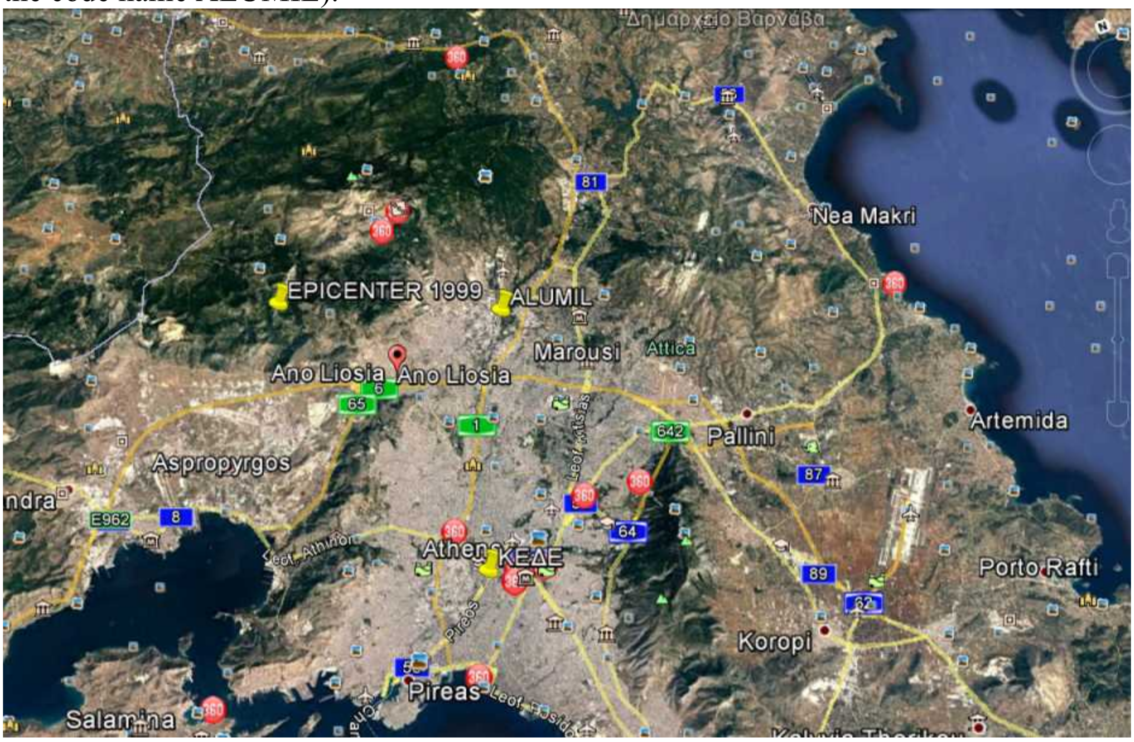


Figure 3. The map of Athens together with the epicenter, the instrument location and the studied building.

Based on the recording of the ground motion with the largest values of the peak ground acceleration for the KEΔE location (record Katagrafi V2), the elastic and inelastic (ductility μ=3) response spectra curves were obtained as shown in figure 4 (left). In the same figure, the corresponding design spectra, Type-1 and Type-2 according to EuroCode-8 provisions for soil ***B*** response modification coefficient ***q=3*** and importance factor 1, based on the design ground acceleration for Athens equal to 0.16*g* are also plotted (***g*** = the acceleration of gravity = 981cm/sec2). As can be seen in these figures, despite the reduction in the spectral response acceleration values by the assumed ductility value (μ=3 to correspond with the assumed response modification coefficient value ***q***=3), the derived inelastic response spectral acceleration values for the horizontal components of the Athens ground motion are approximately 1.5 to 2.0 times larger than the design spectra acceleration values in the period range 0 to 0.3sec. For period values larger than 0.3sec the inelastic response spectra acceleration values are approximately of the same amplitude as the corresponding design spectra acceleration values. This fact indicates the severity of the ground motion and the magnitude of the seismic forces. As already mentioned, the studied building is close to the epicentral region and it is reasonable to expect the level of seismic forces to be even higher than those resulting for the design spectral curves of figure 4 (right).

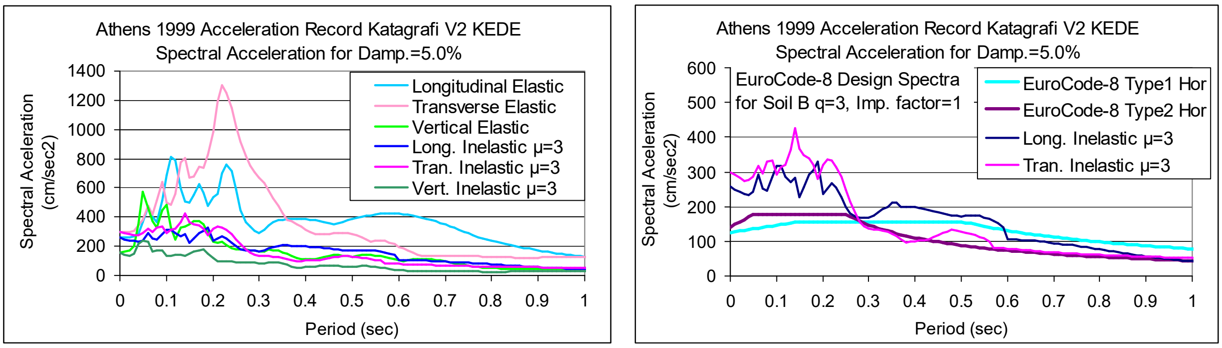
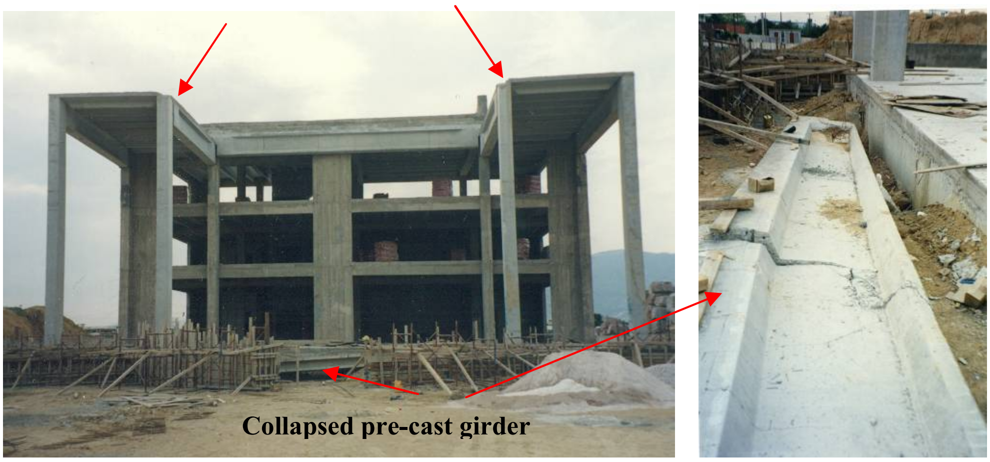


Figure 4. Elastic and inelastic response spectra curves for the KEΔE V2 record of ground acceleration during the Athens 7th of September earthquake sequence together with design spectra.



Figures 5 and 6. Dislocation of a heavy pre-cast girder (B6) at the façade of the office building

The most significant structural damage sustained by this industrial complex was as follows:
a) The dislocation of a heavy pre-cast girder at the façade of the office building (see figures 5 and 6). This girder lost the support at its two ends and fell to the ground (figure 6). In the same location but in the transverse direction the pre-cast girders that formed the 3-D frame

system had signs of distress in their ends that were cracked without, however, loosing their support from the columns below (figure 7).
b) There was additional sign of distress to the bottom of one of the shear walls at the perimeter of the main industrial hall (figure 8).
c) Wide cracking at the supports of certain horizontal pre-cast girders that formed the roofing system of the main industrial hall. This visible cracking developed at the locations where these girders joined the supporting columns. Despite this visible damage none of these girders lost their supports from the adjacent columns (figure 9).

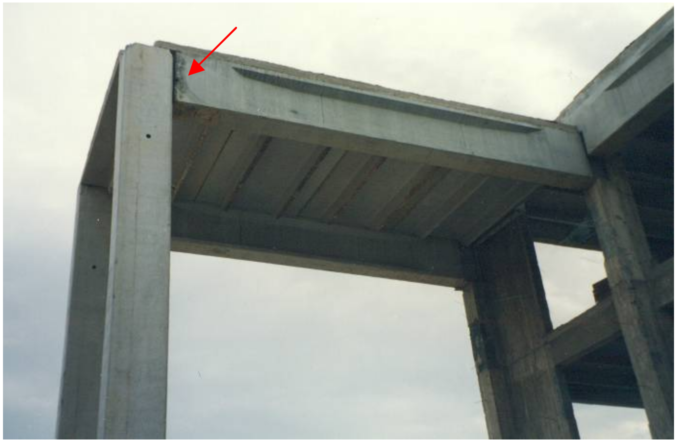


Figure 7. Cracking of pre-cast girder at its left support.

Figure 8. Cracking of shear wall T7

Figure 9. Cracking of the horizontal pre-cast girders that formed the roofing system of the main industrial hall.

a) 

b) 

Figure 10. a) Arial view of the industrial complex (2013), b) main office building entrance. Industrial hall is marked with red and with blue the office and show room area.

2 DESCRIPTION OF THE STRUCTURAL SYSTEM

The total built area is 5.659,13 m2 and it includes the 2 story industrial hall and the 3 story office building with its basement and show room area (see figure 10). The framing system of these two structures, without the office area partition walls and other non-bearing elements such as claddings, glass panel e.t.c., was constructed during 1998 and 1999, just before the September 1999 earthquake. The building owner is Alumil S.A. having as main activity the production of aluminum profiles. The main structural members are pre-cast and pre-stressed elements, which were produced at Preconstructa S.A., Kilkis Greece, located 450km North of Athens and 50km North of Thessaloniki, and then transported to the building site.

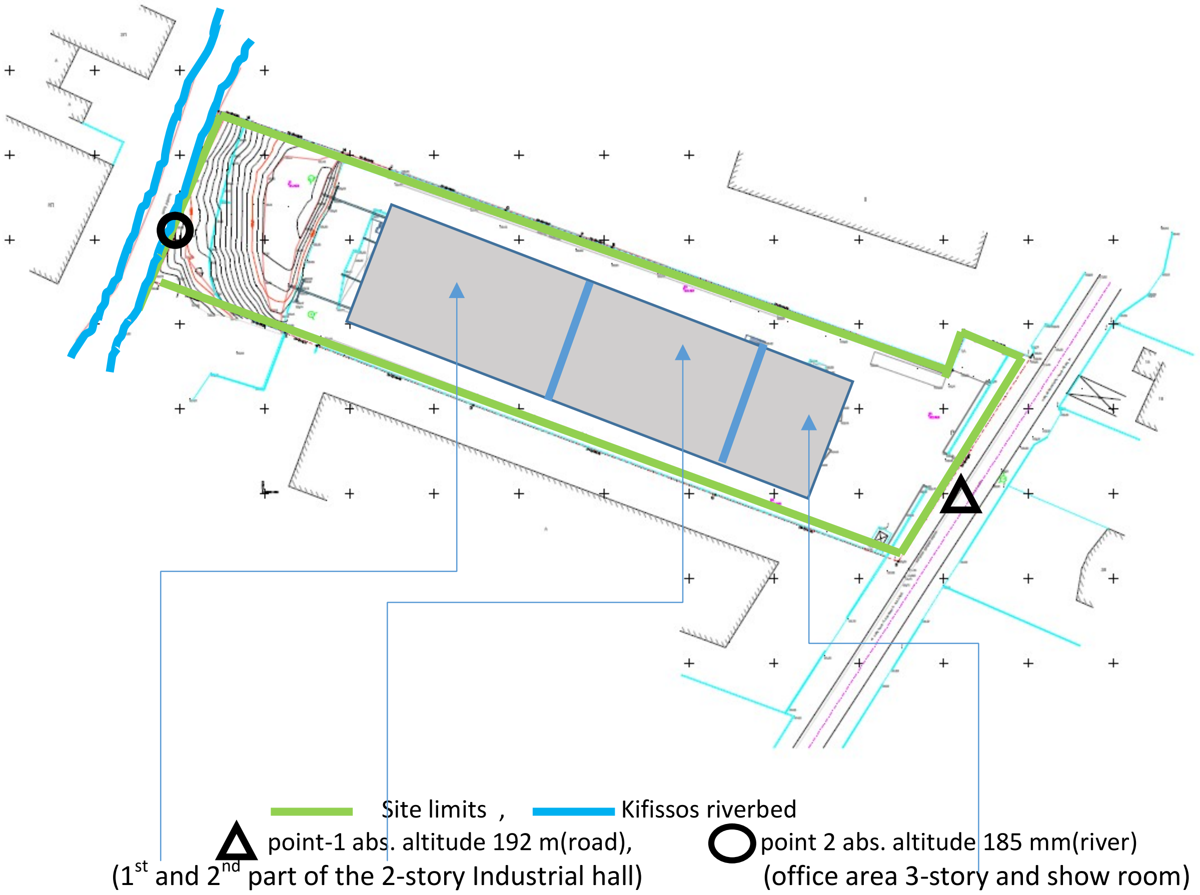


Figure11. Building location at the site (gray box)

Figures 11 and 12 depict the layout of the industrial complex in plan and cross-section, respectively. As can be seen in figure 11, the Kifissos river bed is located at the left side of the built area whereas the right side of the built area, which represents the office building façade, faces the Tatoiou street. In order to build the foundation, a considerable volume of soil was excavated, as depicted in figure 12.

2.1 Structural system of the Industrial Hall

It is a 2-story structure and is constructed from R/C prefabricated and pre-stressed as well as cast in place structural elements. The dimension of each story in plan is 77.24 m x 24.75 m. The total area of the industrial hall is 2x1.911.44=3822.88 m2.

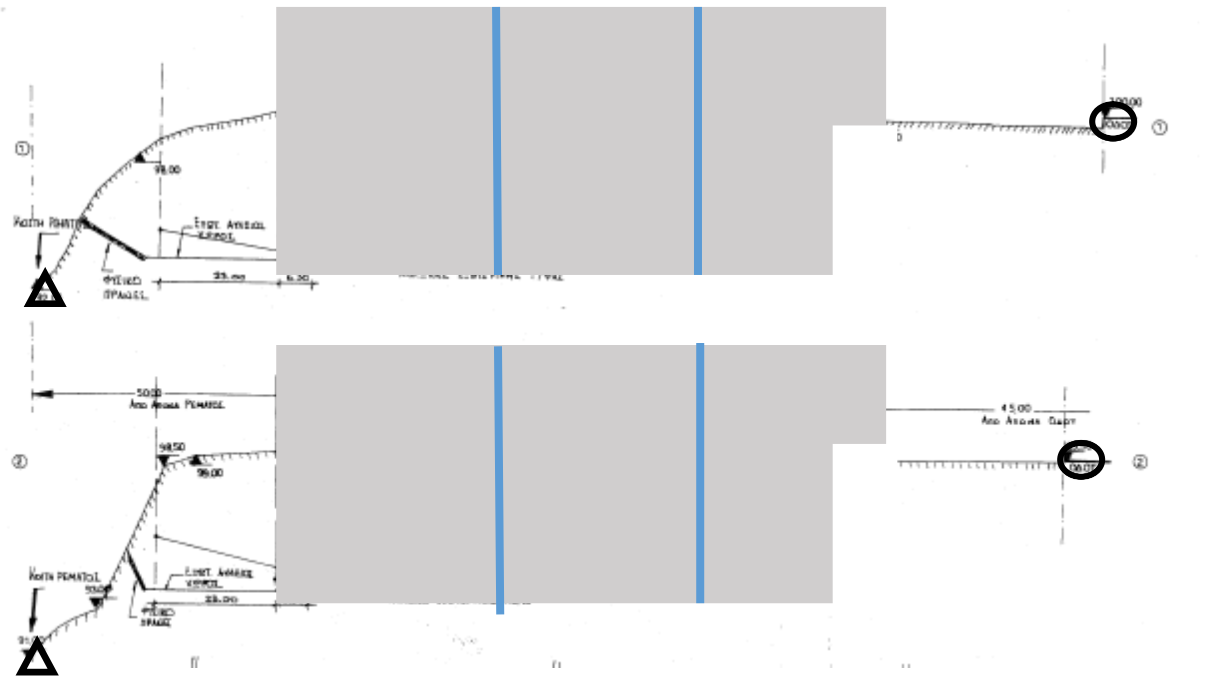


Figure12. Section along the site showing the area that was excavated. The gray boxes denote the building.

-The axis grid is 6,30 m x 8,10 m for the low level (ground floor) and 6,30 m x 24,17 m for the second high level (1st story). The framing system in the transverse direction is a 3 bay frame for the ground floor and a 2 bay frame for the 1st floor.

- Over the foundation level the structure is divided in two similar parts using a construction gap of 50 mm in width. Part of the building is below the ground level. The access to each level is done directly from the surrounding area of the site.

-The total height of the industrial hall is 15,43 m. The ground floor height is 6,15 m from slab to slab. For the second level (1ststory) the clear height under the roof girders is 7,47 m (fig. 13).

-There are two bridge crane lines, one for each level with a crane capacity 30 KN for each line. For the ground floor the crane span is 7,08m from beam to beam (central bay), while for the 1st story the crane span is 23,02 m.

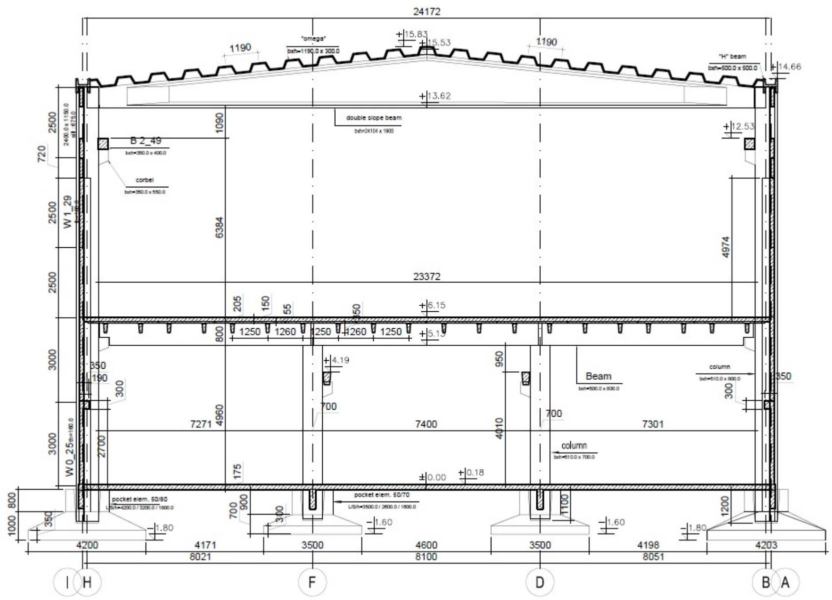


Figure. 13 Typical section of the industrial hall.

-The foundation is constructed using precast pocket elements and precast ground beams. The elements are jointed together using cast in place concrete. A cast in place R/C mat is constructed under the pocket foundation elements. These elements have also extended ties at the bottom surface, which were used for their connection to the concrete mat, as well as for anchor bars at the vertical faces. The large altitude difference (7,0 m) between the main road (Tatoiou) and the river bed dictated the foundation level, which is located approximately 1,00 m. above the river bed (figure 12). In this way the stability of the foundation was ensured.

-The lateral rigidity of the building is achieved by combining pre-cast R/C columns and cast in place shear walls. The pre-cast columns at the perimeter of the industrial hall have been produced in the factory as a monolithic construction, having the full two-story height with a total length of 15,55 m. The columns cross section is 500x800mm (figure 14) for the perimeter axis and 500x700mmfor the inner axis.

- Shear walls constructed with cast in place reinforced concrete (R/C) were placed at the perimeter of the industrial hall in both horizontal directions. They are jointed with the precast columns using ties protruded from the columns. There are eight shear walls four in each main axis direction. The width of each shear wall is 250 mm. The length of the each shear wall, in the major axis is 4000 mm, including the column dimension (figure 15). The height of shear walls with main axis parallel to the building longitudinal direction is 11,00 m while the shear walls parallel to the transverse direction are extended from the bottom to the top of the ground floor minus the precast beam height (figure 20).

- The 1st story floor is constructed form precast beams and single or double “Tee” precast basic slab elements [5]. The main beams are elements, having a cross-section of 500x800 mm (width x height). Each beam has extended ties (close looped bars) at the top, d=10 mm every 150 or 200 mm, embedded in the beam section. The double “tee” precast slab has a width of 2500 mm and its length is 6100 m. The single “tee” slab is of half the width of the double ‘tee’ slab. The basic slab section has a thickness of approximately 55 mm. After the place-ment of the beams and slabs at their final position a R.C. top layer is cast in place with a thickness of 150 mm is added. Thus the total slab thickness is 55+150=205 mm.

- The roof is constructed from pre-cast pre-stressed structural elements. The main girders are tapered of double "tee" double slope shape (figure 13). Their maximum height is 1900 mm and the minimum height is 700 mm. Their length of all these girders is the same equal to 24104 mm. The roof slab is constructed using "omega" pre-cast pre-stressed elements, having a length of 6260 mm a width of 1990 mm and a height of 300 mm. It is of open thin wall section 50 mm thick The transverse roof beams are a "H" shaped pre-cast pre-stressed beams. The "H" beam height and width is 500 mm having a length of 6280 mm (figure 13).

- All the pre-stressed girders and beams are produced in horizontal beds. The main reinforcement is low relaxation wire strands directly bonded to the concrete and anchored using open anchor grips.

- All these beams and girders function as simply supported structural elements. Their connection to their supporting columns is achieved using dowels (steel rods) embedded and extended from the precast columns. The roof girders are supported at the top of the columns using a "Π" shaped "nest" formation.

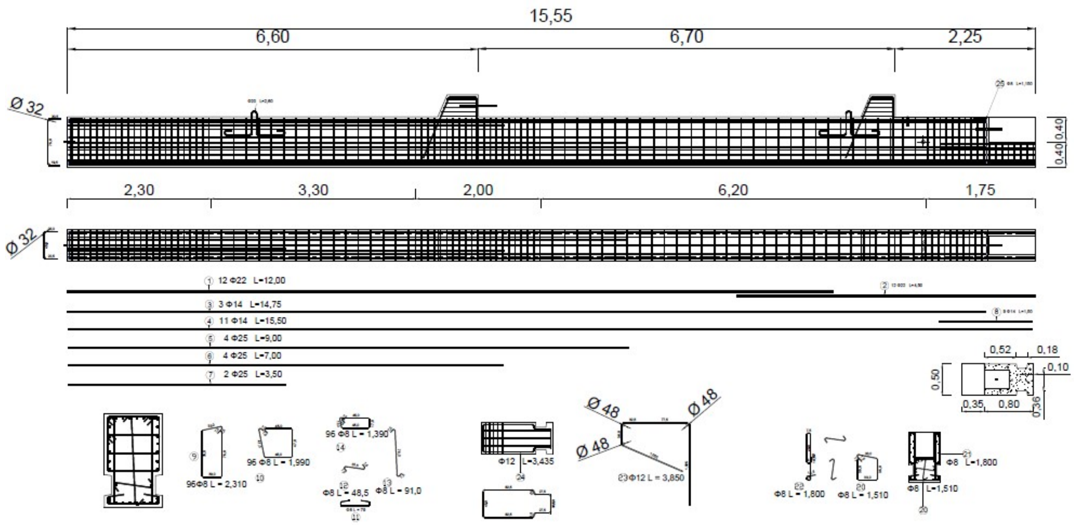


Figure 14 typical 500x800x15550 mm column reinforcement for industrial hall building (shop drawings).

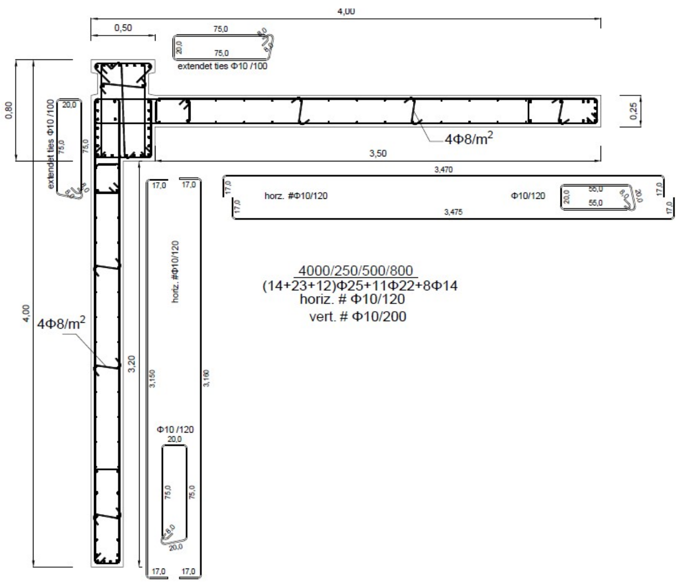


fig.15 precast column 500/800 and cast in place shear walls reinforcement (industrial hall-ground floor-T1).

2.2 Structural system of the Office building

-This is a 3-story structure with a basement. In its front part there is a sun room (show room) which is covered with glass panels. The total height of the office building is 12,32 m and that of the sun room area 11,18 m (fig.16). A structural gap of 60 mm width separates the office building from the industrial hall. The dimensions of the office building in plan is 16,00 x 25,00 m and those of the sun room 9,45 x 25,00 m. The total area is 4x16,00x25,00 + 9,45x25,00 = 1.836,25 m^2. The axis grid is variable from 7,50m to 9,50m for the x axis and from 4,80m to 7,40m for the y axis. For the sun room the y axis grid is 4,80m up to 2x7,40=14,80 m. The roof is flat for either the office building or the sun room area. Over the central sun room area there is a double slope roof made from glass panels.

-The office area is constructed using pre-cast and cast in place R/C members. Those precast members are : double ”tee” basic slab elements for each floor level, 4 columns of the sun room with its pocket foundation elements and the ground beams, double ”tee” pre-stressed elements on the roof of the sun room.

-The cast in place members are the foundation of the office building, the columns, the floor beams and the shear walls (SW). A core wall is used for the elevator shaft.

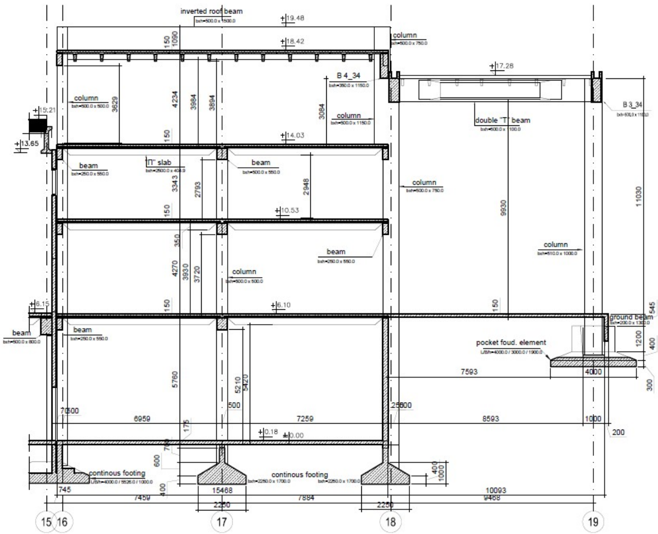


figure16 Section of the office building area.

-The foundation level for the office area is of the same altitude as that of the industrial hall. It is constructed using a continuous footing grid (fig.17). The perimeter of the basement is formed by R/C walls 250 mm thick.

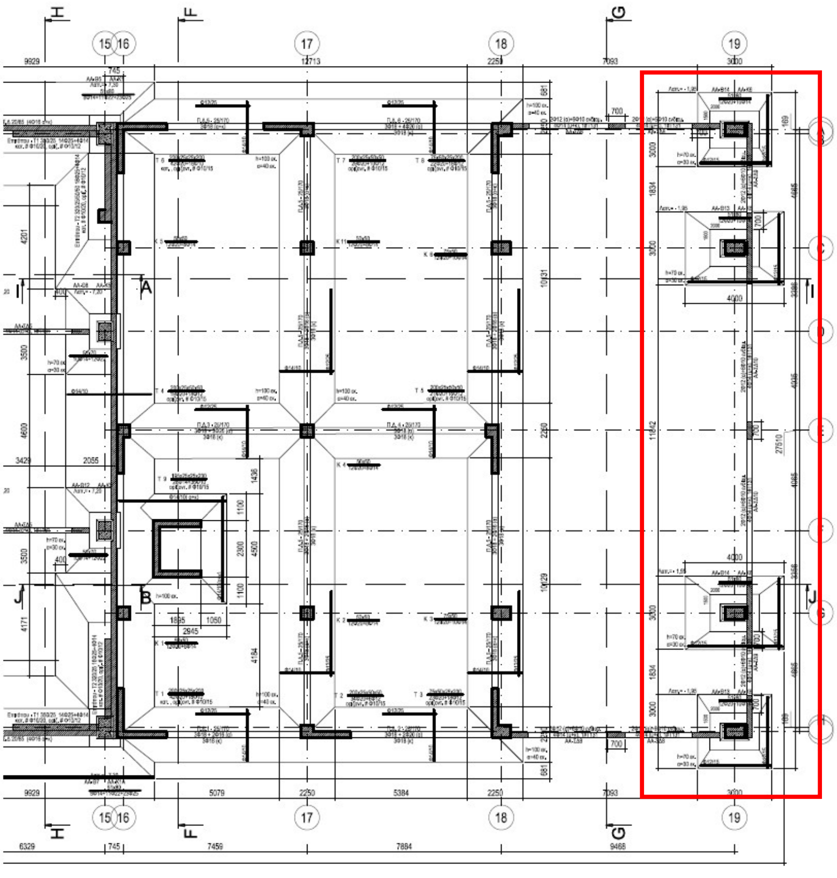


figure 17 Office building foundation level. The part inside the box is founded 5,50 m above the foundation level of the building.

-The framing system for the office building (figure 18) is a combination of beams columns and shear walls. The shear walls are located in the perimeter of the structure except for the elevator shaft core wall. Each wall has a thickness of 250 mm and length of 2,00 m. The column dimensions are 500x500 mm (figure 19) , 750x500 mm and 800x500 mm.

figure 18 office and sun room framing system as constructed before the earthquake.

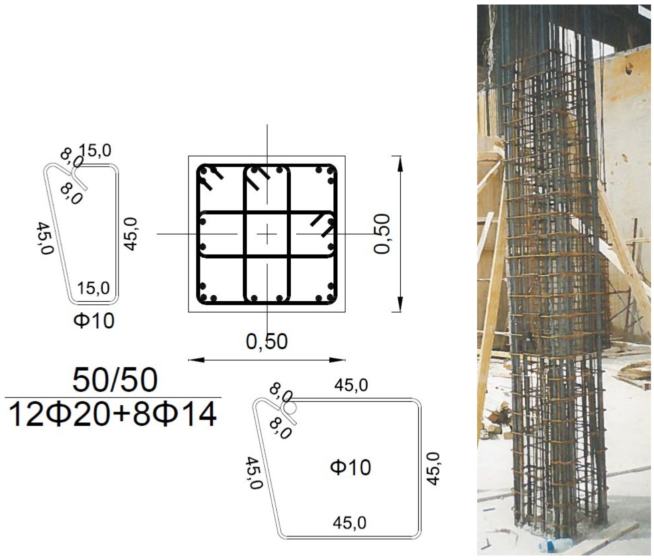


figure 19 column C11 basement

3. NUMERICAL MODELING OF THE STRUCTURE

3.1 Design Loads – Construction Materials Properties

The gravitational load values were according to the general actions on structures of the Greek National Code. The live loads applied on the structure were a uniform load of 7,5 KN/m^2 for the industrial hall 1st level slab and 2,0 KN/m^2 for the building area for all stories except the roof. For both buildings the roof load was taken 0,65 KN/m^2. Dead load values and other partitions weight such as brick walls e.tc. were defined according to the above mentioned code. The moving crane load for the industrial hall was on both levels 30 KN.

The seismic loads were according to Greek National Code edition 1995 as follows:

- A=0,16g (seismic region I according to EAK [9])
- importance factor γi= 1,00
- foundation coefficient θ=1,0 for the industrial hall, 0,90 for the office building.
- soil category B
- characteristic spectrum period T1/T2→0,15/0,60 sec. for the constant value branch of the code design spectrum.
- q factor = 1,80 (≈2,5x75%) for industrial hall and 3,50 for office building.

- soil allowable strength 250 KN/m^2, ks=45.000 KN/m^3.
- Ψ_2 = 0,80 for the industrial hall and 0,30 for the office building.

The concrete quality for the precast elements is:

- Girders and beams in general C30/37.
- “omega” roof elements for industrial hall C35/45.
- double “tee” basic slab elements , columns, pocket found. elements C20/25.
- ground beams C16/20, gladding concrete walls for the industrial hall C30/37.

The topping on the double “tee” elements was C20/25. For the rest cast in place R/C elements the concrete class was C16/20 class. The reinforcing bars for the precast elements were S500 class with yield stress equal to 500MPa. For the office building reinforcement was S400 class with yield stress equal to 400MPa. The wire strands were of S1630/1860 low relaxation class [3],[4].

3.2 Industrial Hall

For the purpose of the gravitational loads analysis each member was provided with its gravitational load according to its actual geometrical properties. The pre-cast pre-stressed beams and “omega” elements were analyzed as simply supported members. The method used is that of limited pre-stressing. The pre-stress losses due to elastic shortening, creep of concrete, relaxation of steel, losses at anchors and due to thermal treatment were taken into account [7]. Each double”tee” basic slab with the topping was also analyzed as simple supported member in order to carry the live load of 7,50 KN/m^2.

For the purpose of the seismic analysis the industrial hall modeling is made using the RSA-PRO software [10]. The computer model includes linear members for the columns, girders and beams and FE shell elements for the shear walls and the 1st level slab members as can be seen in figures 20 and 22.

a) b)

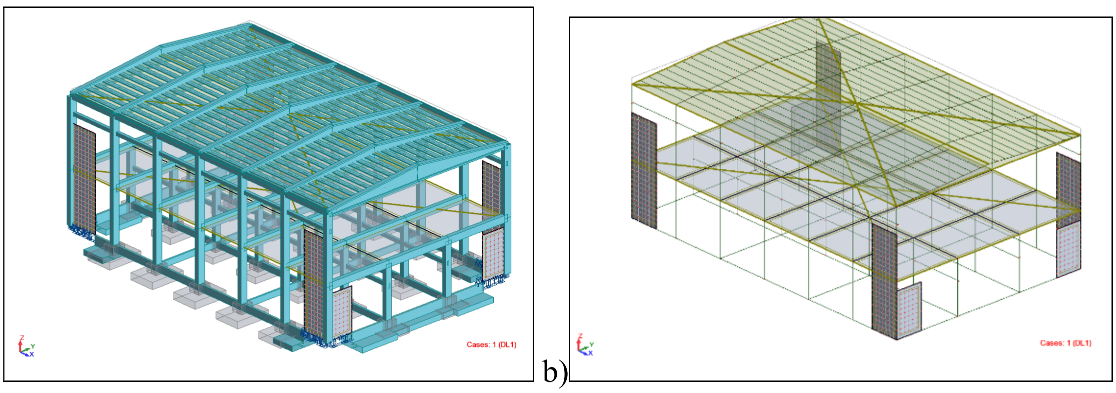


Figure 20. Computer modeling of one part of the industrial hall, a) physical model, b) numerical model with linear frame elements for the columns and girder. 2D planar finite elements were employed for the shear walls. The horizontal diaphragms are indicated by the symbol X and are assumed as planar non-deformable areas.

a) b)

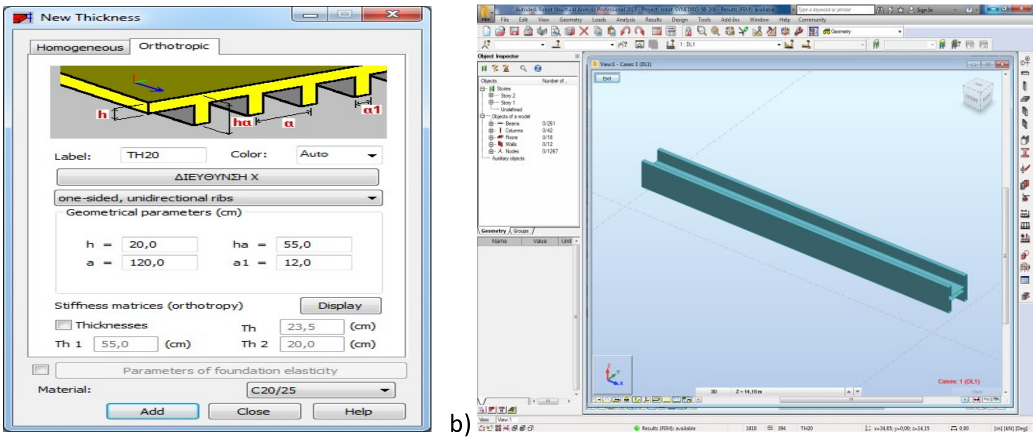


Figure 21. a) double “tee” floor element, b) “H” shaped roof beam.

The double slope double “tee” roof girder was modeled as an “I“ tapered member. The “omega” elements were taken by using an orthogonal member having the same cross section area and weight as the real one. The 1st floor beams are of orthogonal section 500x800 mm. The “H” shaped roof beam was modeled as the actual one by using existing computer linear member (figure 21). Pinned support conditions were chosen for the simply supported pre-cast beam elements. The continuous footing beams and the pocket foundation elements are supported on elastic ground springs using a mean soil coefficient ks = 45.000 KN/m^3 (medium stiffness clay). The floor slabs are assumed to constitute a diaphragm (no mesh generation was made here).

The translation modes obtained from the modal analysis are shown in figure 23. The eigen-period values for the lateral translational modes along the two main axis x, y range between 0,85sec. and 1,04 sec. The 1st mode is the one in the y-y axis (transverse direction) while the 2nd mode is the one in the x-x axis (longitudinal direction). The analysis investigated a) the influence of the rigidity of the roof diaphragm from no diaphragm to the rigid one and b) the influence of the stiffness of the perimeter pre-cast concrete walls (t=160 mm) assumed to act as pinned supported panels (fig.22). This fundamentals period value range is shown in figure 26 with respect to the elastic spectrum of the 1999 earthquake. Note that the value of the response modification factor used in deriving these response spectral curves was equal to q=1.8.

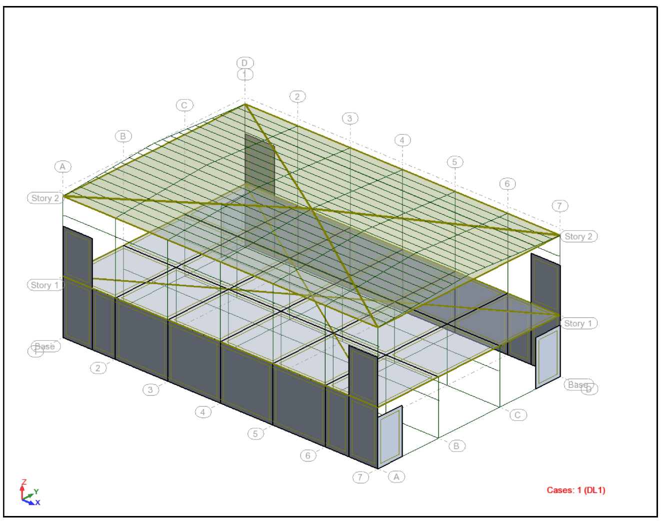


figure 22 perimeter wall pre-cast elements as shell elements

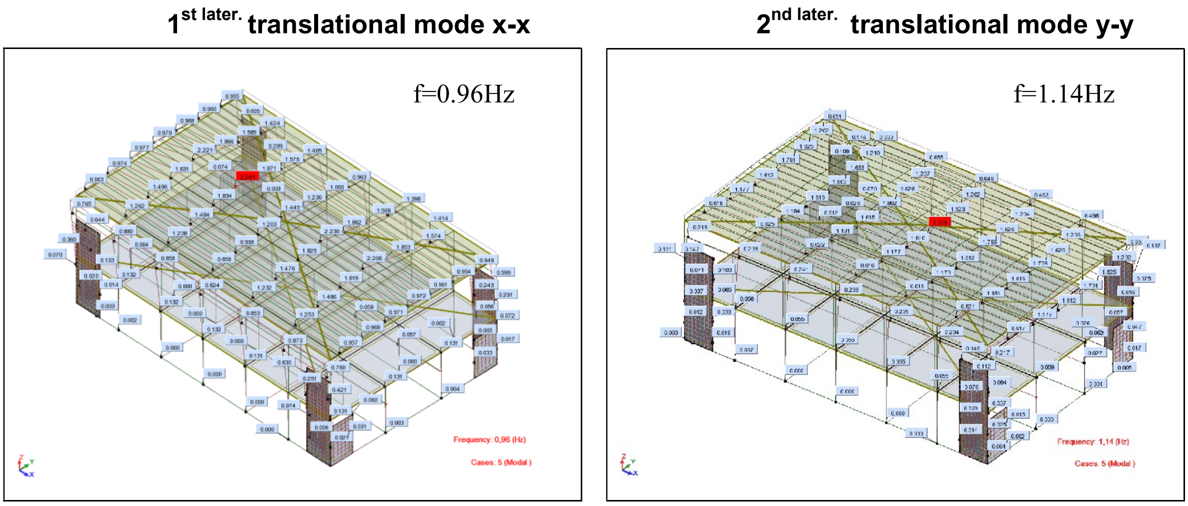


fig. 23 fundamental translational modes in y-y and x-x direction (modal displacements values and frequencies).

3.3. Office Building

The foundation elements are spread footings, continuous footings with elastic ground springs (Ks=45.000 KN/m^3) and ground beams. The basement walls and the upper structure shear walls are modeled using shell elements. The floor slabs are assumed to form rigid diaphragms for each level (fig. 24). The pre-cast roof beams of the sun room have releases at both ends in order to achieve the simple support conditions, which allows rotation about x, y, z axis and restrain the displacement in x, y axis.

The translation modes as obtained from the modal analysis are shown in fig. 25. The eigen-period values for the lateral translational modes along the two main axis x, y range between 0.52sec. and 0.65 sec. The 1st mode is the one in the x-x axis (longitudinal direction) while the 2nd mode is the one in the y-y axis (transverse direction). This fundamentals period value range is shown in fig. 27 with respect to the elastic spectrum of the 1999 earthquake. Note that the value of the response modification factor used in deriving these response spectral curves was equal to q=3.5.

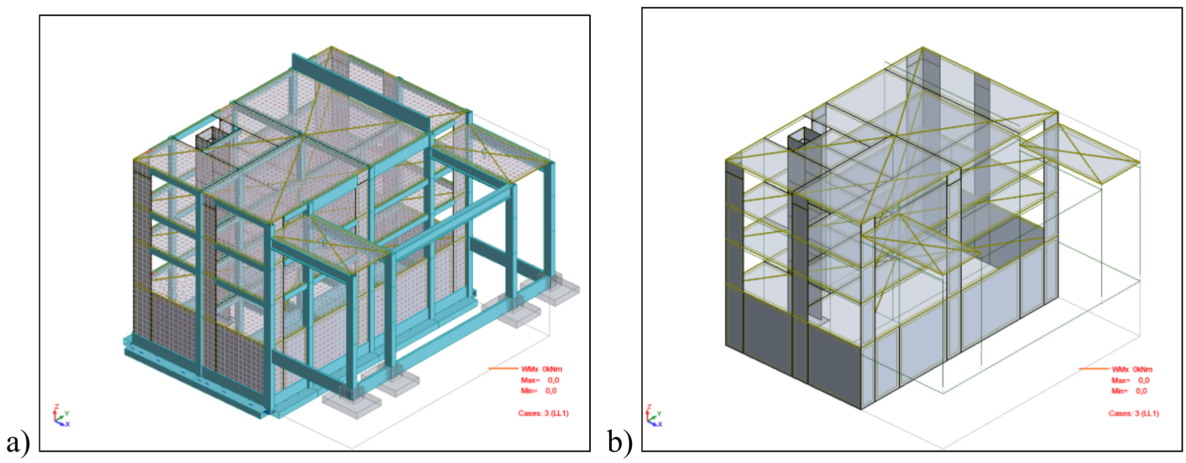


Figure 24. Computer modeling of one part of the industrial hall, a) physical model, b)numerical model with linear frame elements for the columns and girder. 2D planar finite elements were employed for the shear walls. The horizontal diaphragms are indicated by the symbol X and are assumed as planar non-deformable areas.

1st later. translational mode x-x **2nd later. translational mode y-y**

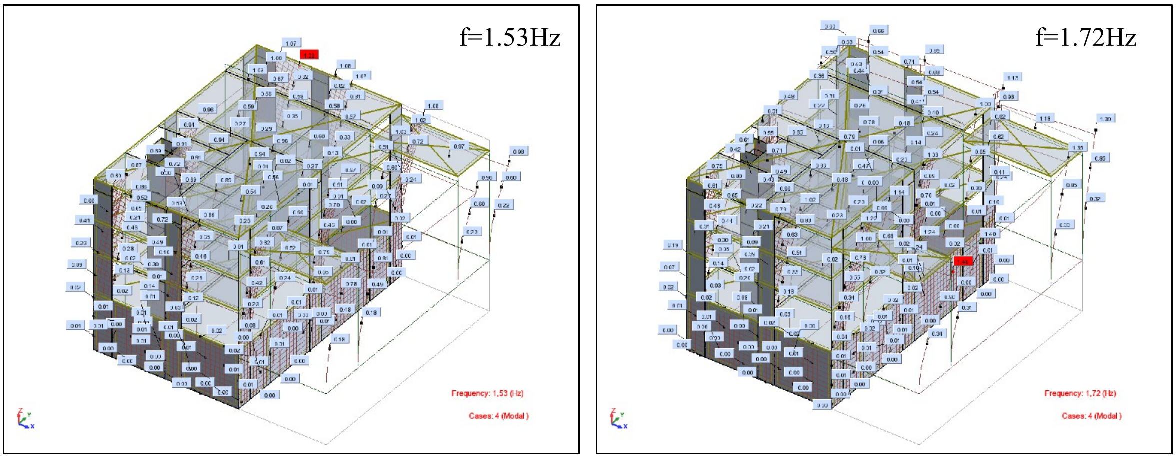


fig. 25 fundamental translational modes in x-x and y-y direction.

3.4. Deriving the levels of the seismic forces.

Based on the preceding numerical simulation of the industrial hall and the office building the eigen-frequency (period) values were obtained for the main translational (x-x and y-y) eigen-modes that mobilize the largest part of the total mass for both structures. In order to show the level of forces that was employed in the subsequent analysis the following procedure is adopted. First, the design EuroCode-8 design spectra that were previously derived in figure 4 for response modification coefficient (q) equal to three (3) now they are derived again. First for the industrial hall for a value q=1.8. This reduced value is dictated from the pre-cast nature of its main-girders and the cantilever-type bending deformation of the vertical columns. On the contrary, a value of the response modification coefficient equal to 3.5 is adopted, because in this case the structural system that resists the seismic forces is a mixed 3-D frame and shear wall system. Similarly, for the industrial hall the inelastic response spectral acceleration curves (longitudinal and transverse) are calculated for a ductility factor equal to 1.8 (μ=1.8), whereas for the office building this value is set equal to 3.5, thus being in line with the corresponding values for the response modification factor (q). In this way the inelastic response acceleration and design spectra curves are derived and plotted in figures 26 and 27 for the industrial hall and the office building, respectively. The relevant eigen-period values for the two main translational eigen-modes (x-x and y-y) are also plotted in the same figures.

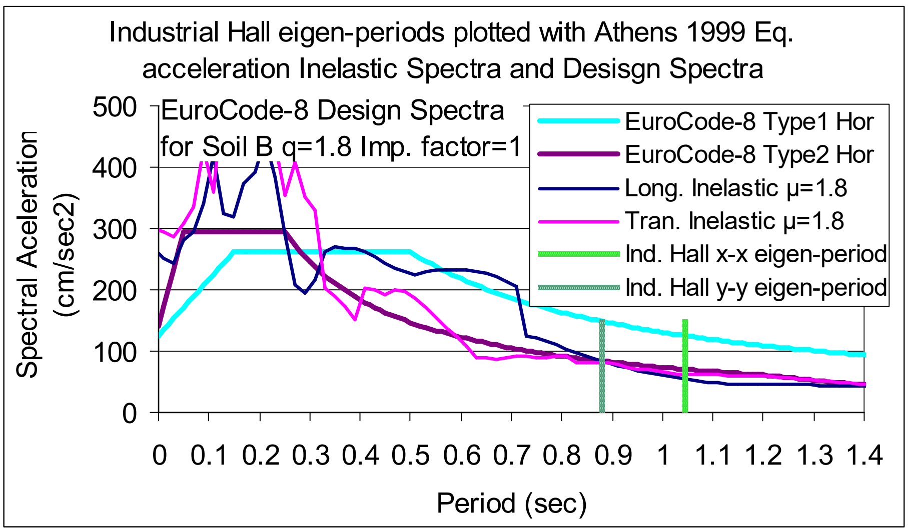


figure 26 response spectra for the industrial hall.

Figure 26 shows that the industrial hall fundamental translational eigen-periods 1st for y-y and 2nd for x-x direction, are out of the range of the maximum ground acceleration. Also the acceleration values from the earthquake seems to be equal or minor from the one used for the analysis. The spectrum values for soil category B according to the EAK period 1995 National Code are approximately the same with the acceleration values derived from the Athens 1999 earthquake.

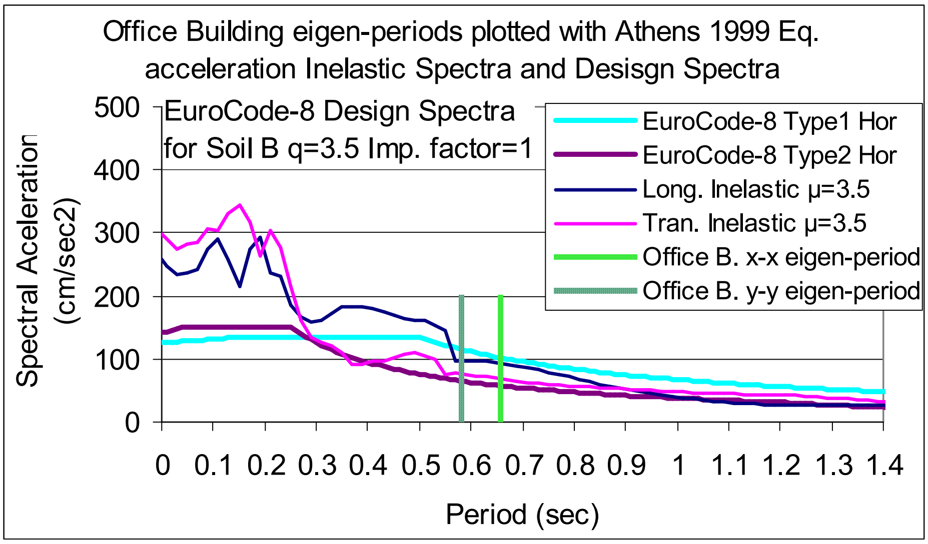


figure 27 response spectra for the office building.

For the office building (figure 27) the range of fundamental translational eigen-periods 1st for x-x and 2nd for y-y direction, are out of the range of the maximum ground acceleration. For this period range the derived earthquake spectrum acceleration are equal or greater from the one used for the analysis.

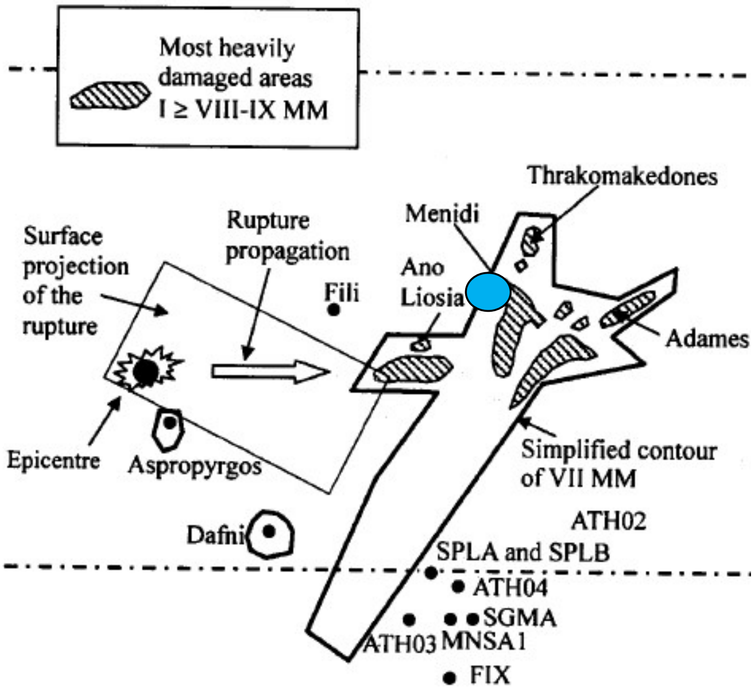


Figure 28. Building location in relation with the most heavily damaged areas [2].

4. OBSERVED BEHAVIOR OF THE STRUCTURE

The location of the office building is shown in figures 3 and 28 in relation to the most heavily damaged areas of the 1999 Athens earthquake. The numerical analysis of the accelerograms during the 1999 Athens earthquake has shown that the seismic intensity exhibit a great dispersion [2]. Due to the different geological conditions both low and high numerical values have been observed for the engineering parameters e.g., PGA varied form 0.085g to 0,511g. For the region of Aharnai, where this industrial complex is located. the estimated PGA was equal to 0,46g.

It must be pointed out that after the earthquake (2003) the seismic design ground acceleration was officially increased by the Ministry of the Environment from 0,16g to 0,24g for areas where most damaged structures were located e.g. Aharnai, Ano Liosia e.t.c.

As already mentioned, the studied industrial complex sustained structural damage mainly to the office building while the industrial hall had only minor structural damage.

4.1 Industrial hall

For a limited number of industrial hall roof girders cracking and spalling of the concrete at the bottom of the cross section near the supports was observed and also inclination from the vertical position for the non-bearing gladding precast concrete walls at the end part of the industrial hall facing the Kifissos river (figures 9, 29). All the precast girders remained in their positions inside the upper precast column “nest” and no dowel damage was observed at their supports. The ground floor level had no damaged structural elements.

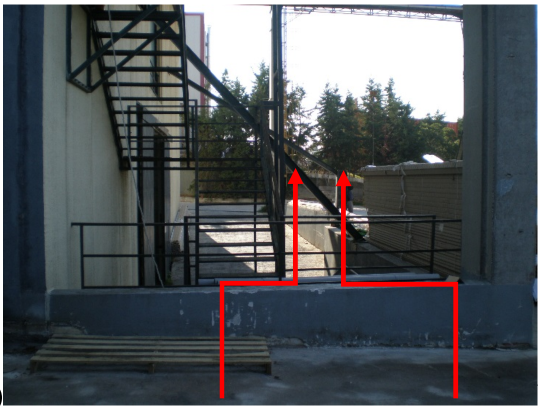
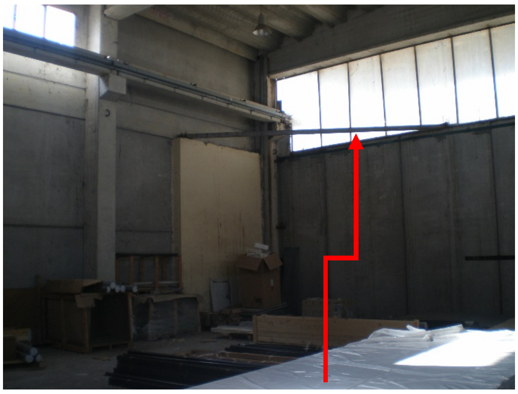
a)  b) 

figure 29 Steel girders used to support on its vertical position the gladding concrete walls after the earthquake from outside and inside of the hall.

4.2 Office building

The office building damages concentrated mainly on the beams located at the perimeter of this building for the 1st 2nd and 3rd floor as well as on a number of ground floor shear walls (at their bottom toe) as shown in figures 5, 6, 7, 8, 30, 31, 32.

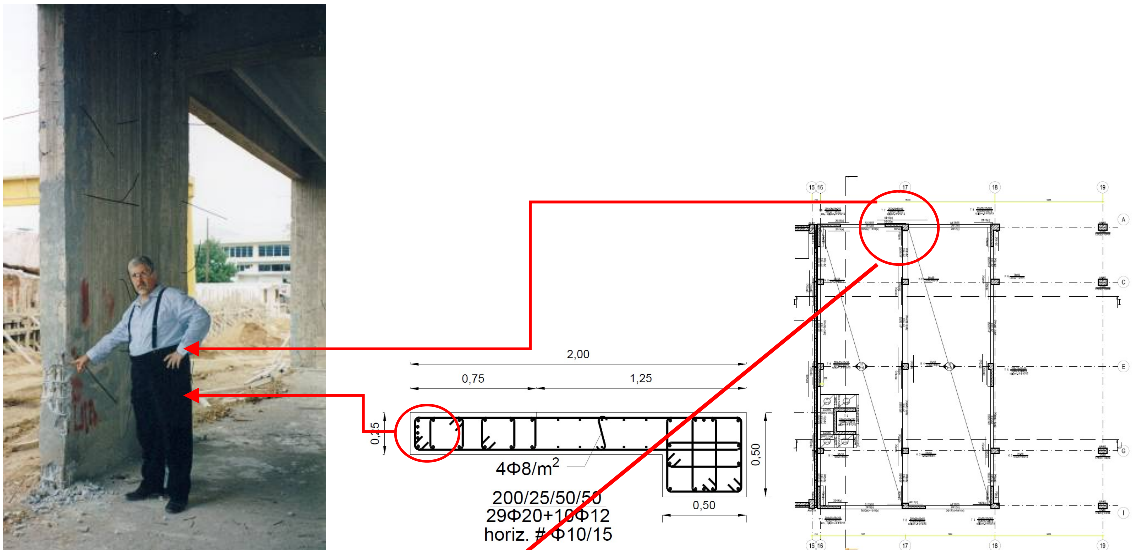


fig. 30 Shear wall T7 at ground floor level and cross-section view at the damaged area.

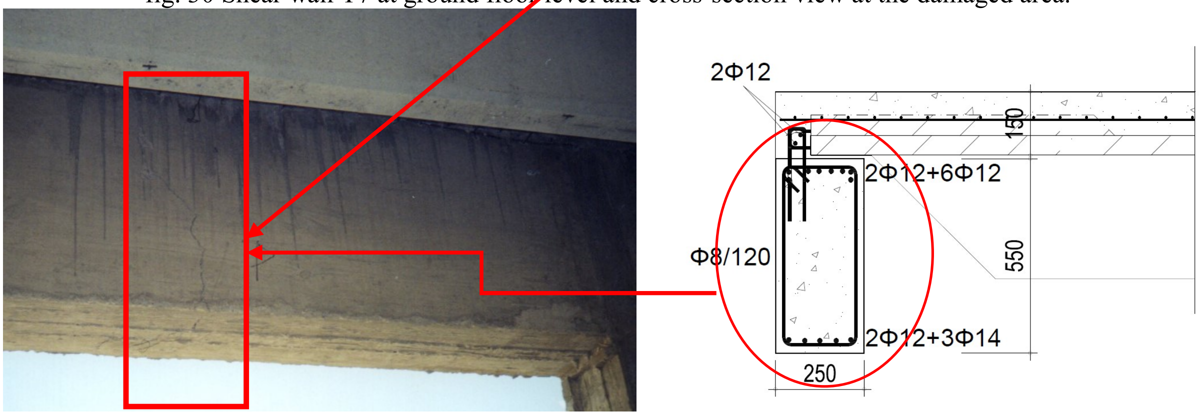


figure 31 cracking of the ground floor ceiling B3 beam 250x550mm at distance ≈ 1,10m from joint B3-T7

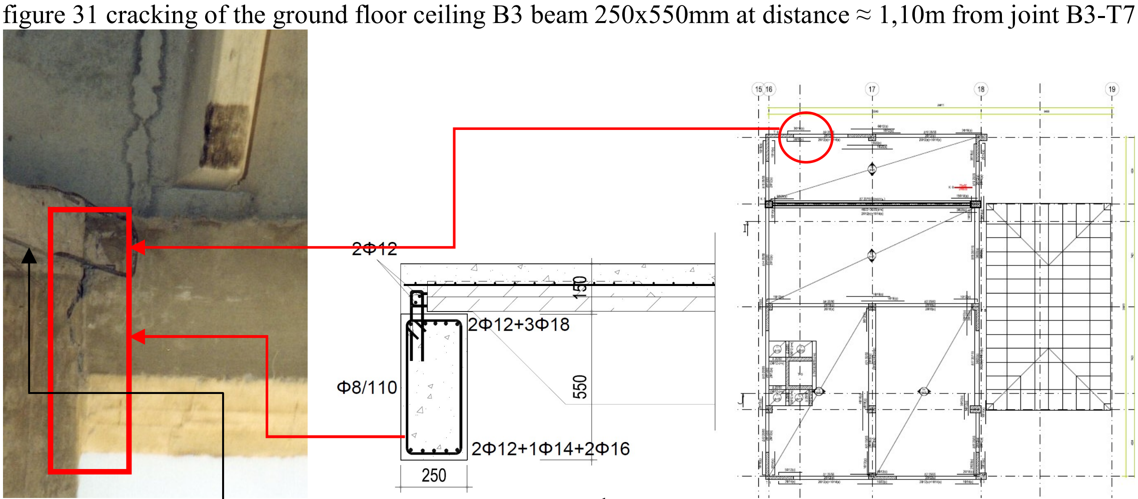


figure 32 cracking at the beam-wall joint (B9-T6) at the 3rd floor level. (B9 250x550mm, stirrups Φ8/110 mm), corbel extended from wall, supporting the precast "tee" slab, not properly constructed.

5. ANALYSIS OF THE OBSERVED BEHAVIOR

The analysis of the observed behavior for the office building structure is conducted using the EAK-95 and EC8 type-1 designed spectrum. For the EC8 design spectrum the seismic coefficient values used are a=0,16g and 0,24g. For the Industrial hall the designed spectrum acceleration values derived using q=1,80 ($q_p \approx K_p$ x q) according to the Greek Precast Code recommendation. The derived spectrum acceleration values for the fundamental translation eigen-period are approximately the same for either EAK or EC8 as depicted in figure 33. Also the response spectrum values of the Athens 1999 earthquake as depicted from figure 26 are in the same value range. This is compared with the minor damage level observed for the Industrial hall. For this reason the analysis is concentrated mainly to the office building.

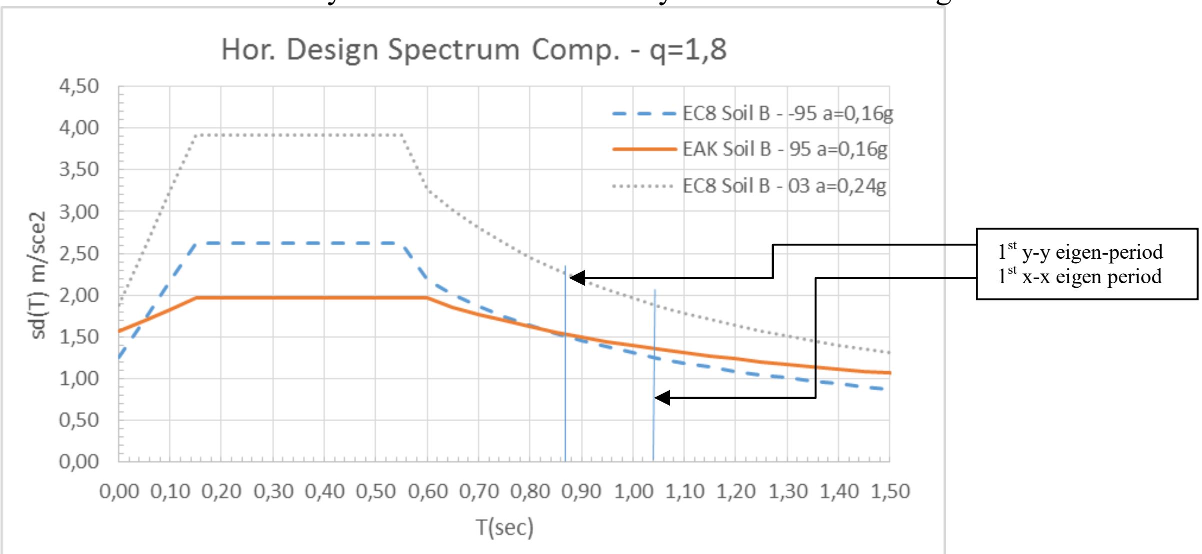


fig. 33 Industrial hall design spectrum according to National Code EAK-95 and EC8-type1 before and after the revision of the seismic coefficient from 0,16g to 0,24g for Aharnai region at year 2003.

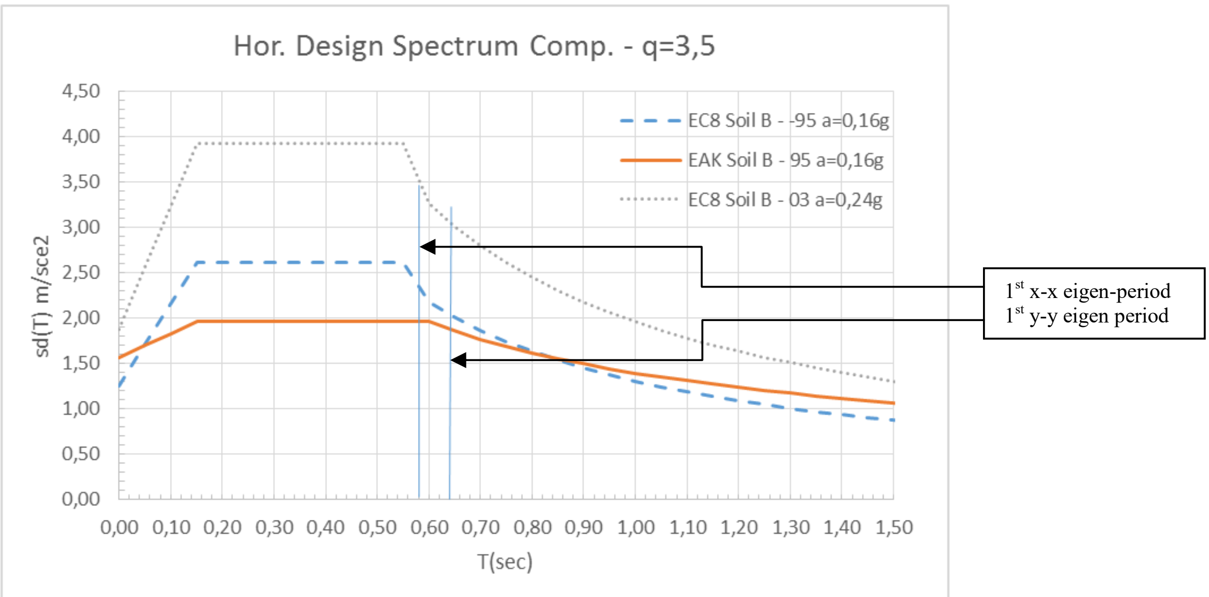


fig. 34 Office building design spectrum according to National Code EAK-95 and EC8-type1 before and after the revision of the seismic coefficient from 0,16g to 0,24g for Aharnai region at year 2003.

The office building structure was analyzed using q=3,5 . According to the EC8 [8],[13] the percentage (n_v) of shear forces resisted by the shear walls is equal to 0,88 which lies within the range 0,65 and 1,00. Therefore, the structural system can be considered as a shear walls type. The relevant relationships and the assumed value for the q factor (equal to 3,5) result in the following, which also applies to the seismic analysis according to the EAK 1995 edition:

qo=3*(au/a1), (au/a1=1,2) and Kw=(0,5<(1+ao))/3<1) [6],[8].

Selected results from such seismic analysis of the office building are presented in what follows, focusing on structural elements which sustained structural damage. These elements are a) The shear wall named T7 depicted in figures 8,30 , b) The beam B3 element at ground floor ceiling (fig.31) , c) The beam B9 at the same vertical position at the building roof (fig.32) and d) The precast girder named B6 on the sun room façade (figs. 5 and 6).

a) **Shear wall T7** _Figures 35, 36 and 37 present the maximum bending moment diagram, bending moment capacity and maximum shear force diagram, respectively.

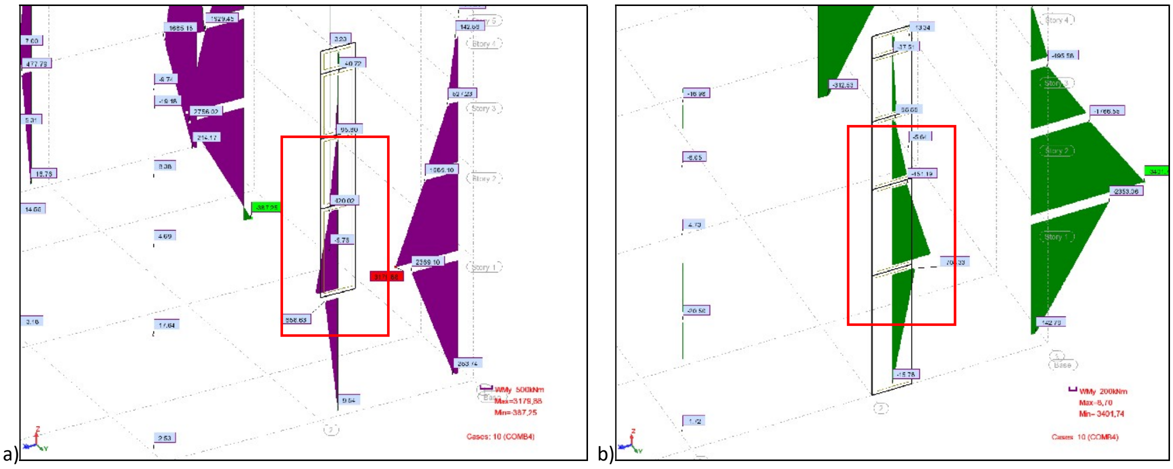


Table 1 max. and min values of the above diagrams along the major shear wall axis

Shear Wall T7 Max, min bending mom.	My(1) (kNm) ag=0,16 EAK	My(2) (kNm) ag=0,16-EC8	My(3) (kNm) ag=0,24-EC8	My(2)/My(1)	My(3)/My(1)
max My	1157,10	1426,63	2129,60	1,23	1,84
Min My	-1119,31	-1388,84	-2091,81	1,24	1,87

Figure 35 max. bending mom. of shear wall T7 at ground level a)EAK ag=0,16 b) EC8 ag=0,16g (office build.) , M_{ed} = 3.261,40 KNm << Mrd_{max}.=3.261 KNm.

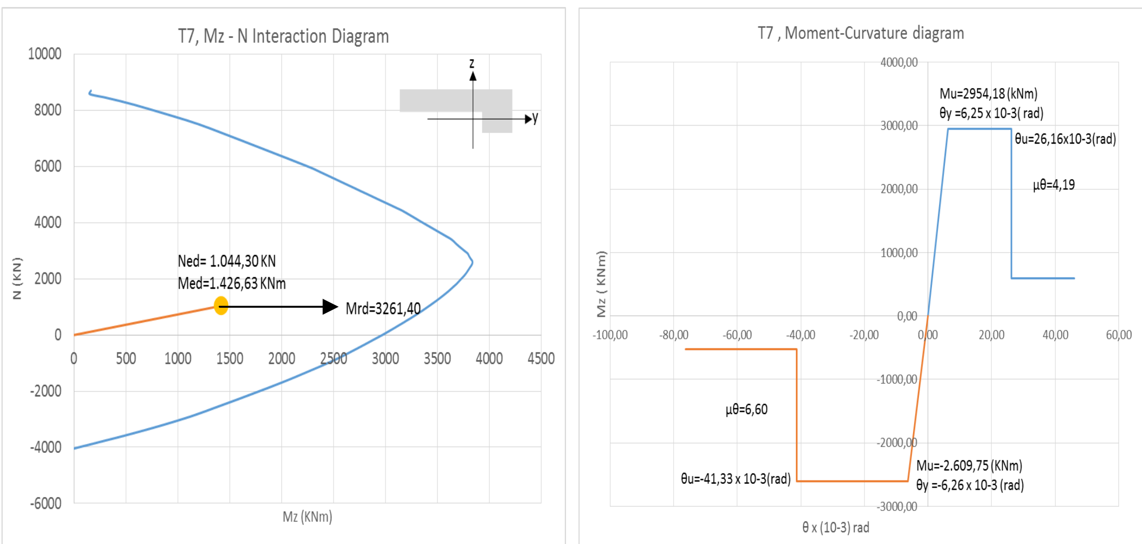


Figure 36 Shear wall T7 ,M-N interaction and moment-curvature diagram (EC8-95ag=0,16g). Mz major axis.

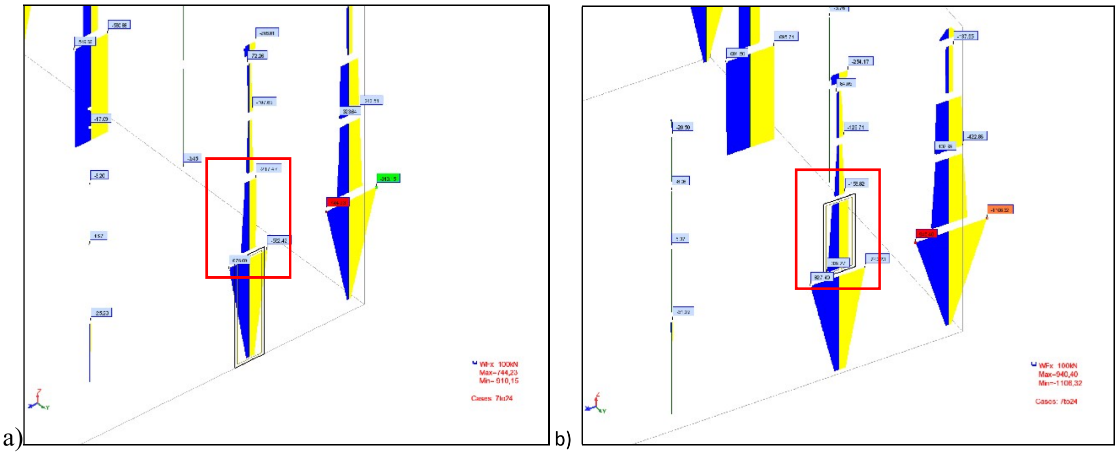


Table 2 max. and min values of the above diagrams along the major shear wall axis

Shear Wall T7 Max. min Shear forces	Vy(1) (kNm) ag=0,16 EAK	Vy(2) (kNm) ag=0,16-EC8	Vy(3) (kNm) ag=0,24-EC8	Vy(2)/Vy(1)	Vy(3)/Vy(1)
Max. Vy	282,86	339,27	489,36	1,20	1,73
Min Vy	-205,43	-261,84	-411,92	1,27	2,00
Crv = Ved/Vrd	Vcd/Vrd3≈1,0	Ved/Vrds=1,3		-----	-----

(EAK-95, Vcd = 760 KN, Vrd3=740 KN), (EC2,8 edge column Ved = 246 KN, Vrds=189 KN).
figure 37 max. shear forces of shear wall T7 at ground level a)EAK ag=0,16 b) EC8 ag=0,16g (office build.), $c_r = V_{cd}/V_{rds} = 1{,}30 > 1{,}00$ for EC8-0,16g.

The bending capacity of the shear wall as designed and constructed is adequate to withstand the maximum bending moment demands as derived from this analysis (figures 35, 36). However, this is not the case for the shear capacity of this wall (figure37, table 2). Using the EC8 type-1 spectrum values the shear capacity for shear wall T7 is exceeded by the corresponding shear demand by approximately 30%. The corresponding design and capacity values for the given geometry and reinforcement of the cross section are derived using the Tol-Raf software [11].

b),c) Beams B3 and B9.

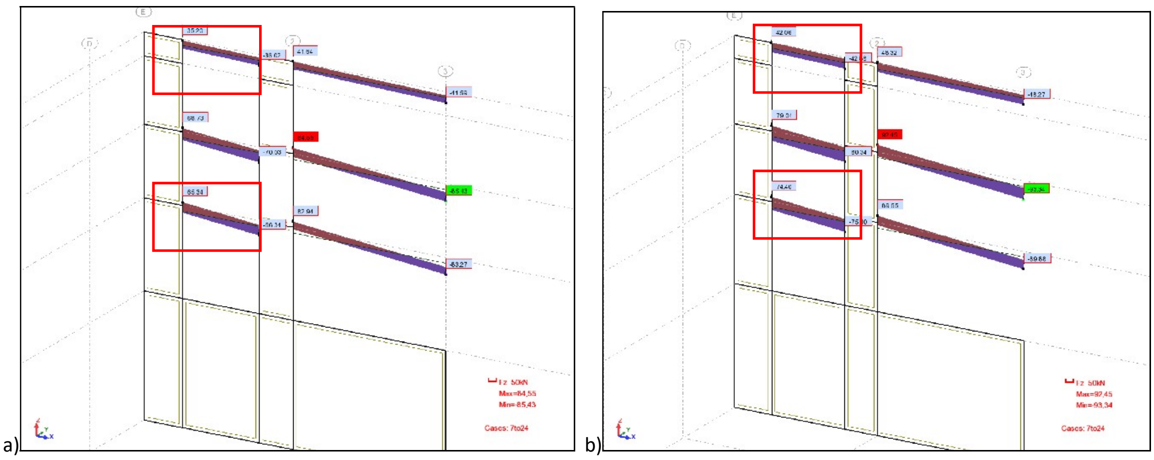


figure 38 max shear force of beam B3 at ground level and B9 at roof a)EAK ag=0,16 b) EC8 ag=0,16 (office building)

Table 3 max, min values of shear force and bending moments of the above diagram

Beam B3 / Shear Force	Vz(1) (kN) ag=0,16 EAK	Vz(2) (kN) ag=0,16-EC8	Vz(3) (kN) ag=0,24-EC8	Vz(2)Vz/(1)	Vz(3)/Vz(1)
Max,min Vz	65,34	74,40	98,01	1,14	1,50

Beam B3 / Bending Moment	My(1) (kNm) ag=0,16 EAK	My(2) (kNm) ag=0,16-EC8	My(3) (kNm) ag=0,24-EC8	My(2)/My(1)	My(3)/My(1)
Max. My Min. My	62,60 -90.28	80,59 -108,09	127,45 -154,46	1,28 1,20	2,03 1,71

Beam B9 / Shear Force	Vz(1) (kN) ag=0,16 EAK	Vz(2) (kN) ag=0,16-EC8	Vz(3) (kN) ag=0,24-EC8	Vz(2)/Vz(1)	Vz(3)/Vz(1)
Max. min. Vz	35,20	42,06	59,93	1,42	1,70

Beam B9 / Bending Moment	My(1) (kN) ag=0,16 EAK	My(2) (kN) ag=0,16-EC8	My(3) (kN) ag=0,24-EC8	My(2)/My(1)	My(3)/My(1)
Max. My Min. My	55,12 -60,68	68,76 -74,29	104,37 -109,34	1,25 1,22	1,89 1,80

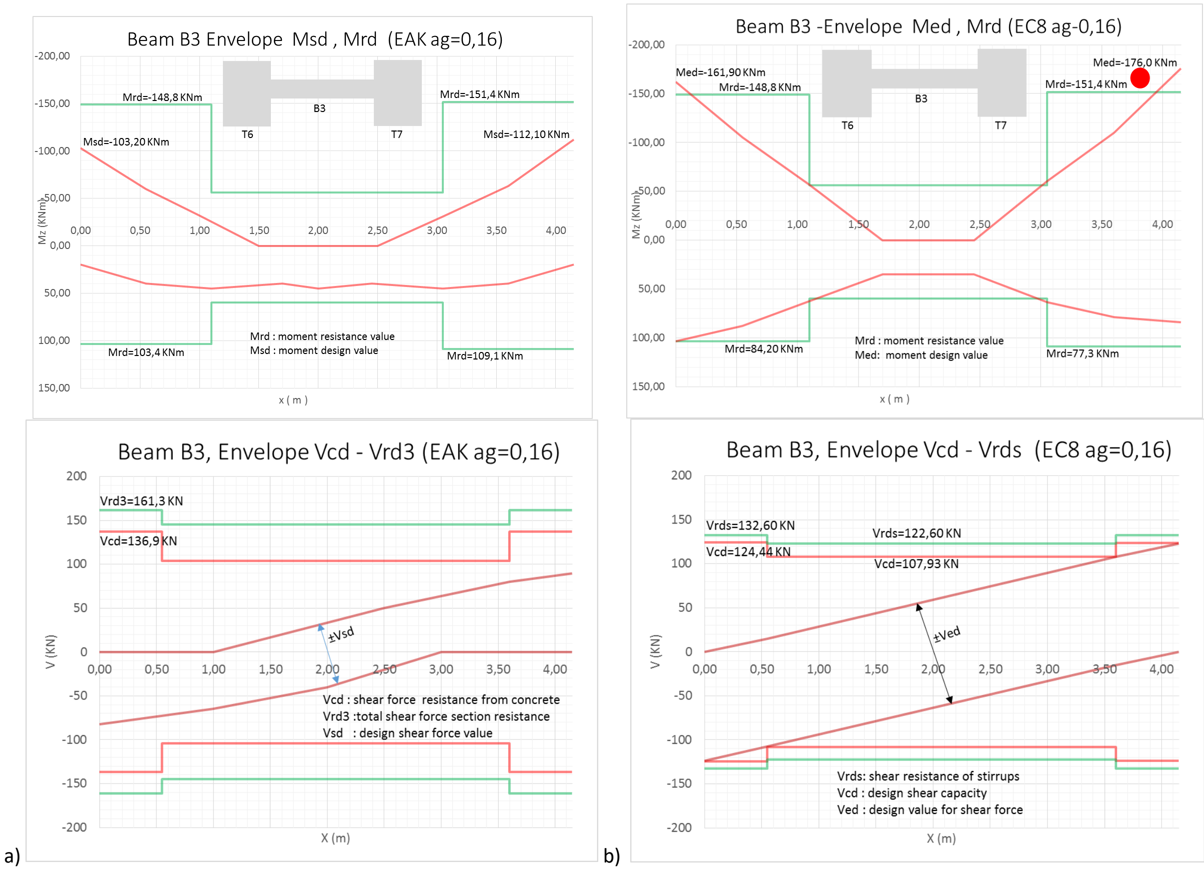


figure 39 Bending moments and shear forces envelops (design loads/capacity) for B3 ground floor beam , a)EAK-95(a=0,16g) and b)EC8 (Type-1, a=0,16g), $C_r(M_{ed}/M_{rd})$=1,16>1,00 - $Cr(V_{ed}/V_{rds})$≈1,00.

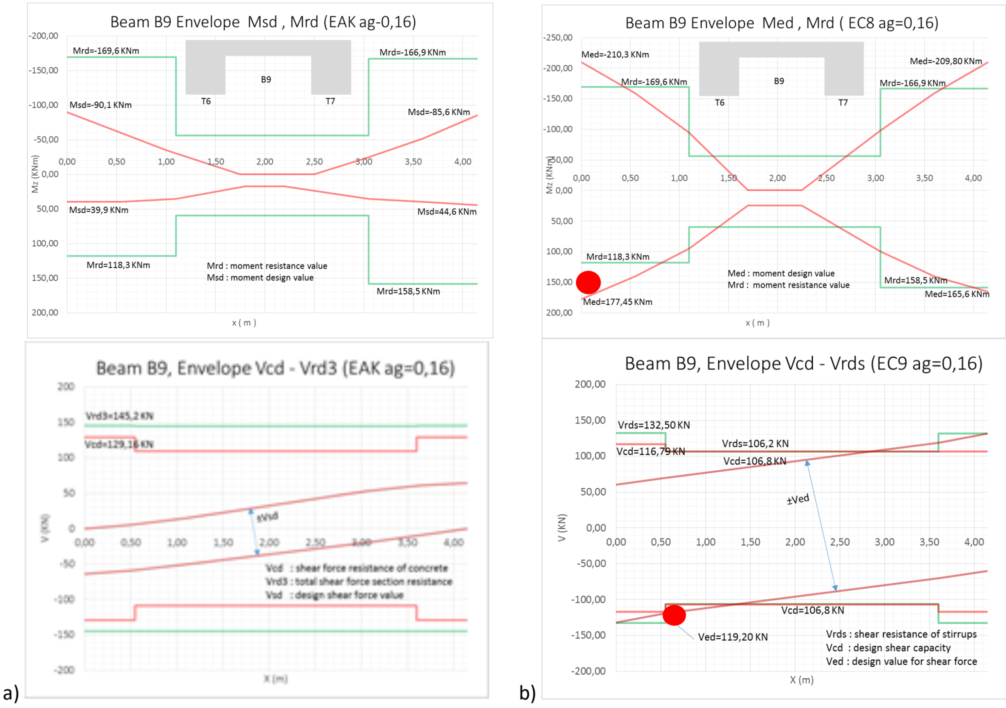
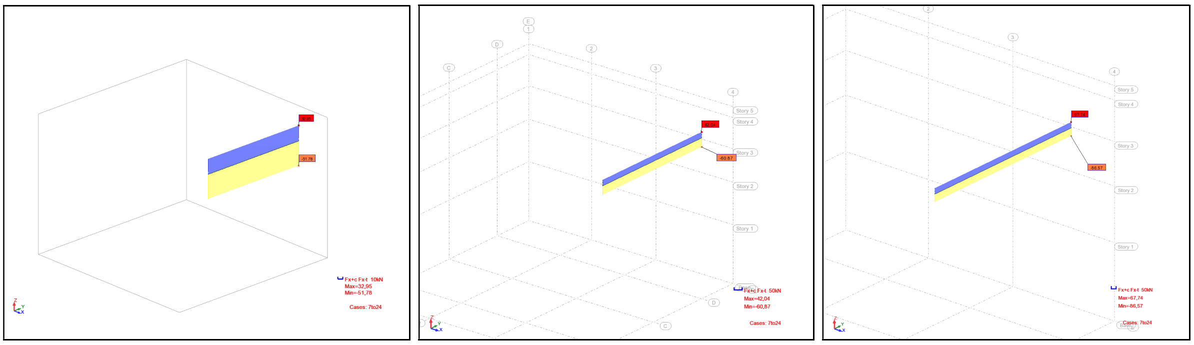


fig.40 Bending moments and shear forces envelops (design loads/capacity) roof beam B9 , a)EAK-95(a=0,16g) and b)EC8(Type-1, a=0,16g), $C_r(M_{ed}/M_{rd})$=1,50 - $Cr(V_{ed}/V_{rds})$≈1,20

d) precast beam B6 connection to C6.



Fx=51,78 KN,EAK(a=0,16g) Fx=60,87 KN,EC8(a=0,16g) Fx=86,57KN,EC8(a=0,24g)

figure 41 Precast shear force at beam column dowel for beam/column B6 – C8 connection (sun room ceiling).

BEAM - COLUMN DOWEL CONNECTION
COLUMN AAK6

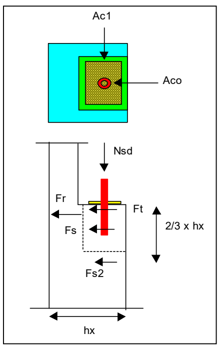
CONCRETE C 20 | STEEL B 500 C
γc = 1,5 | γs = 1,15
fcd = 13,3 N/mm2 | fyd = 434,8 N/mm2

COLUMN DIMENSIONS
dx = 0,8 m
dy = 0,5 m

Support Area
hx = 0,3 m
hy = 0,3 m
e = 0,25 m

ELASTOMERIC-PIECE
SPEBA SERIES 3120 (σd = 15 N/mm2)
ax = 250 mm
ay = 250 mm
t = 20 mm
N = 938 KN

APPLIED LOADS
Nsd = 68,68 KN
Vsd = 60,87 KN



1. Check for condrete upper face compression
(0) = failure
(1) = o.k.

ef = 42 mm
b* = 250 mm
a* = 167 mm
Aco = 41.667 mm2
Ac1 = 65.000 mm2
F_{Rdu}= 694 KN > Nsd = 68,68 KN O.K.
F_{Rdu1}= 1.833 KN > F_{Rdu}= 694 KN

2. Check for dowel capacity
(0) = failure
(1) = o.k.
B 500
fyd = 434,8 N/mm2

dowel d = 28 mm 1 τεμ.
δ = 1,04
c1 = 140,0 mm >=< 84 mm
c2 = 150,0 mm >=< 140 mm
Du,d = 56,6 KN < Vsd = 60,87 KN FAILURE

figure 42 dowel capacity for the steel rod connection B6 and C6 [12].

From the preceding selected analysis results for the office building it can be concluded that:

Shear Wall T7

- The bending moment capacity of shear wall T7 is larger than the corresponding demands even the ones resulting from the maximum spectrum values, employing seismic design ground acceleration a= 0,24g, as can be seen from figures 35,36.
- The shear capacity of wall T7 is exceeded by the shear force demands obtained using the spectrum values of EC-8 type-1 Crv=1.30 (figure 37 table 2) and employing seismic design ground acceleration equal to 0,16g, which is quite close to the recorded spectral acceleration of Athens 1999 earthquake.

Beams B3 and B9

- The B3 beam bending moment capacity is exceeded by the demands obtained using the EC8-type1 spectrum values having a capacity ratio Crm=1,16>1,00 (figures 39), while the shear force capacity is approximately equal to 1,00. This seems to comply with the observed behavior of a cracked section for this beam at the critical region end (figure 31).
- The B9 bending moment and shear force capacity is exceeded by 50% and 20% by the corresponding bending moment and shear force demands, respectively (figures 40).

Beam B6 (sun room ceiling)

- The pre-cast beam B6 at the roof of the office building (figs. 5, 6) was the only one that was constructed without a "nest" connection at the top of the precast column. The force applied to the dowel (d=28 mm as build), as resulted from the analysis using EC8-type1 design spectra and seismic design ground acceleration 0.16g (60.87KN), exceeds the capacity (56.6KN) of the steel rod (figure 42). As already mentioned, this beam was not supported by a "Π" shaped "nest" whereas the roof girders of the industrial hall were supported by such "Π" shaped "nests". Thus, it can be concluded, on the basis of this analysis and the relevant observations, that the "nested" connection of the girders to the columns exhibited up to a point satisfactory performance for this intense earthquake ground motion. On the contrary, the performance of the beam-to-column connection that did not employ these "Π" shaped "nests" was unsatisfactory and should be avoided figures 5,6).

6. CONCLUSIONS

a. The Industrial hall having the fundamental eigen-period outside the peak design spectrum values and using a conservative value for the q factor equal to 1,80, according to the Greek Precast R/C Building Code recommendations, has shown a rather satisfactory behavior during the Athens 1999 earthquake. The use of "Π" shaped "nests" for the connection of the main roof precast-prestressed girders with the supporting columns prevented them from dislocating or overturning. Moreover, the use of such a connection reduced the forces sustained by the dowels at this connection detail leading to a satisfactory performance for the level of forces generated from this earthquake sequence. On the contrary, the absence of such a "nested" connection for the office façade girder B6 caused the dislocation and overturning after the dowel failure. From the observed unsatisfactory performance of the beam-to-column connection that did not employ these "Π" shaped "nests" it can be concluded that such connection detail should not be used in seismic regions. The used seismic analysis was capable at predicting this type of dowel failure.

b. The employed numerical analysis for the office building structure resulted in bending moment and shear force demands for the critical structural elements that when compared with their corresponding bending moment and shear force capacities are well in agreement with the observed behaviour of these structural elements during the Athens 1999 earthquake. Using the EC8 type-1 spectrum values, which was shown to be closer to the Athens 1999 earthquake acceleration response spectra, the shear force capacity of the shear wall T7 is exceeded, while the bending moment capacity is greater the corresponding demand.

c) The value for the q factor equal to 3,5, which was used in the seismic design of the office building, proved to be a non-conservative on the basis of the observed structural damage that this structure sustained during the Athens 1999 earthquake. Although the moment ductility ratio $\mu\theta=4,2$ is achieved based on the outcome of the seismic analysis, the shear capacity deficiency seem to governs the behavior of the shear walls thus producing a rather non-ductile performance.

d) The revision of the seismic code that resulted in an increase of the seismic design ground acceleration by 50% (from 0,16g to 0,24g) for the region that sustained considerable structural damage during the Athens 1999 earthquake (2003) seems to be in the right direction.

e) The present investigation demonstrated that the use of pre-cast reinforced concrete members, when properly designed and detailed, can be employed with confidence in seismic regions. Towards this end the composition of the structural system, utilizing shear walls with the appropriate bending moment and shear capacities, as well as effective interconnections of the pre-cast structural elements and the cast in-place structural members are the key factors for a safe structural performance.

ACKNOWLEDGMENTS

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