THE SEISMIC BEHAVIOUR OF A PRE-CAST R/C INDUSTRIAL COMPLEX SUBJECTED TO THE 1999 ATHENS-GREECE EARTHQUAKE

George C. MANOS1, Demetrios MPOUFIDIS2, Th. ZAFEIRIOU3

ABSTRACT

The dynamic and earthquake behavior of a pre-cast industrial complex, located at the epicentral area of the Athens 1999 Earthquake, is studied. It is 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. Shear walls in both horizontal directions are added in its perimeter, cast in place and connected to the pre-cast structural columns. The foundation is also formed by pre-cast pocket elements and ground beams which are interconnected with cast in-place parts. The three-story office building 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 also pre-cast. 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 observed 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 use of pre-cast reinforced concrete members, when properly designed and detailed, can be employed with confidence in seismic regions.

Keywords: Pre-cast structural members; Industrial buildings; Observed earthquake behavior; Numerical simulation; and Athens 1999 earthquake.

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 (Saitta et al 2012). 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. In the following sections the dynamic and earthquake behavior of an industrial complex is presented, located in Aharnai-Athens and subjected to the strong ground motion of the Athens 1999 Earthquake (Manos 2011). The industrial hall is 77,24m x 24,75m in plan with 7,47m

1 Emeritus Professor, Ex-Director Lab. of Strength of Materials and Structures, Dept. Civil Engineering, Aristotle University, e-mail: gcmanos@civil.auth.gr
2 Dr. Civil Engineer, Lab. of Strength of Materials and Structures, Aristotle University, e-mail: dboufj@otenet.gr
3 Civil Engineer, Postgraduate student, Lab. of Strength of Materials and Structures, Aristotle University, e-mail: tmzafi@hotmail.com
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 Manos 2011) as well as to many residential buildings.

Figure 1. Unsatisfactory performance of pre-cast industrial buildings due the displacement and strength limitations of the flexible connections of the pre-cast horizontal structural elements. (Saitta et al. 2012)

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

This industrial complex was under construction during the Athens, Greece Earthquake, as can be seen from figures 5 and 6 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 KEAE in Figure 3, Anastasiadis 1999) 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 indicated in figure 3 together with the exact location of the studied building complex (yellow pin with the code name ALUMIL). Based on the recording of the ground motion with the largest values of the peak ground acceleration for the KEΔΕ location (record Katagrafi V2), the elastic and inelastic (ductility \( \mu = 3 \)) 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 0.16g 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 (\( \mu = 3 \) 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 3. The map of Athens together with the epicenter, the instrument location and the studied building.](image3)

![Figure 4. Elastic and inelastic response spectra curves for the KEΔΕ V2 record of ground acceleration during the](image4)
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 (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.
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. The framing system of these two structures, without the office area partition walls and other non-bearing elements such as claddings, glass panels, etc., 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, produced by Preconstructa S.A., Kilkis Greece, located 450km North of Athens and 50km North of Thessaloniki, and transported to the building site.

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. More details are given by Mpoufidis et al. (2017). Figure 10 depicts a cross section of the industrial hall.

- The 1st story floor is constructed form precast beams and single or double “Tee” precast basic slab elements (Philipps and Sheppard 1980). 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 placement 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 10). Their maximum and minimum heights are 1900 mm and 700 mm, respectively. 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 10).

- 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 10. Typical section of the industrial hall.
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 (figure 11). 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 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. More details are given by Mpoufidis et al. (2017). Figure 11 depicts a cross section of the office building. The following is a summary description of this structure.

- The office area is constructed with pre-cast and cast in place R/C members. The 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 sun room roof.
- 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.
- 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. The perimeter of the basement is formed by R/C walls 250 mm thick.
- 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 2.0 m. The column dimensions are 500x500 mm, 750x500 mm and 800x500 mm.

3. NUMERICAL MODELING OF THE STRUCTURE

Various numerical simulations of the structural system for either the office building or the industrial hall were employed. Initially, in one group of numerical simulations all the pre-cast structural elements were considered as being connected through elastic hinges with infinite axial stiffness to the rest of the structural system. In another group of numerical simulations an alternative approach was followed in connecting the pre-cast structural elements to the rest of the structural system. This time the flexibility of each connection was studied in some depth. Next, the connection of the pre-cast structural element was done through 3-D link elements of small length but with finite stiffness in all six degrees of freedom. These link elements were provided with elastic properties that approximate their behaviour that included elastic as well as inelastic parts. Details of the first group of numerical simulations can be found in the work by Mpoufidis et al. (2017). Selective results of this work will be presented also here together with the results from the alternative approach whereby the connection of the pre-cast elements was addressed with 3-D link elements of finite stiffness, as already mentioned before. In both cases the following loading conditions are considered:

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/m2 for the industrial hall 1st level slab and 2.0 KN/m2 for the building area for all stories except the roof. For both buildings the roof load was taken 0.65 KN/m2. Dead load values and other partitions weight such as brick walls were defined according to the above mentioned code. The moving crane load for the industrial hall was on both levels 30 KN.

For the seismic loads either Euro-code 8 or the Greek Seismic Code were employed as follows: a) Ground design acceleration either $A=0.16g$ (seismic zone I) or $A=0.24g$ (seismic zone II) according to the Greek Seismic Code. This was necessary due to a revision in the seismic zonation introduced in
region where this industrial complex is located in 2000. This revision took place after the 1999 Athens earthquake sequence and holds since then. b) Importance factor \( \gamma_i = 1.00 \), foundation coefficient \( \theta = 1.0 \) for the industrial hall, \( 0.90 \) for the office building. c) \( q \) factor = 1.80 (≈2.5x75%) for industrial hall and 3.50 for office building. d) Soil allowable strength 250 KN/m², \( k_s = 45,000 \) KN/m³. e) \( \Psi_2 = 0.80 \) for the industrial hall and 0.30 for the office building.

The concrete quality for the precast elements is: a1) Girders and beams in general C30/37. b1) “omega” roof elements for industrial hall C35/45. Double “tee” basic slab elements, columns, pocket foundation elements C20/25. c1) Ground beams C16/20, gladding concrete walls for the industrial hall C30/37. d) 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. e) 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.

3.1. Numerical simulation of the office building

In this case the stiffness of the connection of the pre-cast structural elements was derived from the relationships that are included for the steel inserts in the Greek code for the repair and strengthening of R/C structures. The load-displacement variation of such connections is depicted in figure 12. For transferring shear loads through the connections of the pre-cast elements of the office building, employing steel bars of 28mm diameter, the elastic stiffness that results applying the procedures of this code for cyclic loads is approximately equal 312KN/mm. The floor slabs of the office building are assumed to form rigid diaphragms for each level. The main translation modes that resulted from the numerical simulation are depicted in figures 13a and 13b. In the subsequent dynamic spectral analysis the first fifteen (15) eigen-modes were employed which mobilized 93% and 92% of the total mass in the x-x and y-y directions, respectively. A number of acceleration spectral curves were employed in the analysis. These are depicted in figure 14. Note that a value of the response modification coefficient equal to \( q = 3.5 \) is adopted for the office building because in this case the structural system that resists the seismic forces is a mixed 3-D frame and shear wall system. In this way the inelastic response acceleration curves, based on the Athens 1999 recorded
ground acceleration, and design spectra curves based on the Euro-code formulas are derived and plotted in figures 14. The relevant eigen-period values for the two main translational eigen-modes (x-x and y-y) found from the numerical simulation of the office building are also plotted in the same figure. As can be seen from these figure the spectral acceleration for the office building derived from either Athens ground motion 1999 record inelastic spectral curves or the Type-1 Euro-code design spectral curves for peak design ground acceleration 0.16g are quite similar in the period range of the main translational modes for the office building.

The maximum amplitude in this case is approximately equal to 11% g (g is the acceleration of gravity equal to 981cm/sec2). However, for peak design ground acceleration equal to 0.24g the Type-1 Euro-code design spectral curves in the period range of the main translational modes for the office building have amplitudes larger than the spectral accelerations derived from the Athens 1999 record. The maximum spectral amplitudes in this case are approximately equal to 18% g.

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Figure 15. Shear wall T7 at the ground floor.

Figure 15 depicts the shear wall at the ground floor of the office building which exhibited signs of distress after the 1999 Athens earthquake. The flexural demands at the base of this wall, as resulted from the dynamic spectral analysis using the spectral curves of the Athens 1999 record have maximum and minimum values equal to 1516KNm and -1577KNm, respectively. Given the axial compressive force condition that is of the order of -900KN the resulting flexural capacity is well over 3000KNm,
which signifies safe flexural performance for wall T7. On the contrary, the shear demand for the same loading case has maximum and minimum values equal to 304KN and -341KN, respectively. The comparison of these values to the T7 shear capacity attributed to the shear reinforcement of T7, which according to EN 1992-1:2004 is equal to 188KN, indicates a possible shear limit state. This is in agreement with the observed behaviour (figure 15). For the demands resulting when the seismic zone II peak design ground acceleration is considered, the shear limit state will be reached much faster.

Figure 16. The left support of beam B9 at the 3rd floor.

Using again the acceleration spectral curves of the Athens 1999 record, the flexural demand for this beam at this location is 66KNm and -58KN against a capacity of 51KNm, which again signifies flexural limit state. Moreover, the shear demand 71KN and -54 KN also exceeds the capacity signifying shear limit state. These observations also agree with the observed performance. Finally using yet again the acceleration spectral curves of the Athens 1999 record the relevant shear transfer demand of 71.3KN is obtained of the connection of the pre-cast beam at the façade of the office building (see figures 5 and 6) with the side R/C frames. This demand is compared to the shear transfer capacity curve of figure 12 with the plot of figure 17; in this plot d signifies the relevant slip displacement in the direction of the applied shear force (Fu). An agreement with the observed performance is seen.

Figure 17. Comparison of shear transfer demand with the corresponding capacity of the connection of the façade pre-cast beam.

3.2 Numerical simulation of the industrial hall

This time the force displacement relationship of the connections of the pre-cast structural elements with the rest of the structural system was derived from a 3-D non-linear simulation that included the non-linear behaviour of the concrete volume of the steel tie as well as of the bond that governed the interaction at the interface between the steel tie and the surrounding concrete volume. As already mentioned, the numerical simulation of this connection was done through a 3-D link element having properties for all six (6) degrees of freedom. The stiffness properties of this link for each one degree of freedom was found by imposing the relevant displacement and rotations corresponding to each degree of freedom and studying the resulting non-linear behaviour of the connection through this 3-D non-linear simulation. The geometric constraints that defined each particular connection were also taken into account in this 3-D non-linear simulation. This is depicted in figures 18a and 18b for the imposed translations along the z and the y axes, respectively. Next, the obtained in this way non-linear behaviour for the pre-cast members’ connection was utilized in the 3-D simulation of the industrial hall. This was done in the following two ways. For the linear dynamic spectral analysis the initial
approximately linear part of the force-displacement behaviour of the connection for each degree of freedom was used as elastic stiffness properties of the 3-D link elements that numerically simulated these connections. Alternatively, when a non-linear “push-over” analysis was employed the full non-linear behaviour of each degree of freedom of this connection, as shown by figures 18a and 18b was provided to govern the relevant degrees of freedom of the 3-D link elements.

![Figure 18a. Non-linear behaviour of the force-displacement behaviour of the studied connection for the imposed translational displacement along the z axis.](image)

![Figure 18b. Non-linear behaviour of the force-displacement behaviour of the studied connection for the imposed translational displacement along the y axis.](image)

Apart from the behaviour of the connections of the pre-cast structural elements with the rest of the structural system which was numerically simulated in the way described above the influence of the pre-cast infill panels was also numerically simulated. These infill panels were placed in all the bays of the R/C frames at the perimeter of the industrial building and their stiffness contribution was approximated using the methodology that is described in the work by Manos and Soulis (2012). In this way the resulting equivalent truss elements were provided with stiffness properties which had a linear elastic and an inelastic non-linear part. Again, the former was used in the linear dynamic spectral analyses whereas both parts were use in the subsequent non-linear “push-over” analyses. The main translation modes that resulted from the numerical simulation are depicted in figures 19a and 19b.

![Figure 19a. Eigen-period 0.99sec y-y translation](image)  Figure 19b. Eigen-period 0.94sec y-y translation
In the subsequent dynamic spectral analysis the first five (5) eigen-modes were employed which mobilized 99% and 94% of the total mass in the y-y and x-x directions, respectively. A number of acceleration spectral curves were employed in the analysis. These are depicted in figure 20. Note that a value of the response modification coefficient equal to q=1.8 is adopted for the industrial hall. This reduced value is dictated from the pre-cast nature of its main-girders and the cantilever-type bending deformation of the vertical columns. In this way, the inelastic response acceleration curves, based on the Athens 1999 recorded ground motion, and design spectra curves based on the Euro-code formulas are derived and plotted in figures 20. The relevant eigen-period values for the two main translational eigen-modes (x-x and y-y) found for the industrial hall are also plotted in the same figure.

The flexural demands at the base of the main corner shear wall of the industrial building are indicated in figures 21a and 21b together with the corresponding capacities, when the dynamic analysis employed the spectral curves of the Athens 1999 earthquake record. As can be seen in this figure the flexural capacities are well above the corresponding demands, signifying a safe performance even for the higher demands of the Euro-Code Type-1 design spectral curves for design ground acceleration 0.24g (see figure 20). This type of analysis also resulted in shear force transfer demands and the connections of the pre-cast members below the capacity found as indicated by figures 18a and 18b thus also signifying a safe performance. These remarks are in agreement with the observed performance during the 1999 Athens earthquake sequence.

4. CONCLUSIONS

- 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 pre-cast / pre-stress 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.

- 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.

- 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. From the flexural behaviour a sufficient ductile performance is ensured (moment ductility ratio $\mu_\theta=4.2$). However, the non-ductile shear behaviour governs the performance of the shear walls.

- 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.

- 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. To-wards 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.

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6. REFERENCES


