Seismic analysis and design of a precast concrete framed structure

A. G. Vlassis[1], C. C. Spyrakos[2] & S. Tsoukantas[2]
[1]*Department of Civil and Environmental Engineering, Imperial College, UK*
[2]*Department of Civil Engineering, National Technical University of Athens, Greece*

Abstract

This study presents the analysis and design of a single-storey industrial building planned to serve as a repair and maintenance facility for the Greek Railway Organization (GRO) near Athens. The structural system is a precast concrete frame. Several issues regarding modelling and analysis of the structure are identified and discussed. In order to increase the stiffness in one of the two orthogonal directions of the structure, diagonal steel braces interconnecting the upper part of the concrete columns are used. Two sets of analysis are performed to investigate the effect of the braces on the structural response. It is shown that seismic behaviour can be significantly improved with the addition of the new members. Furthermore, dynamic analysis is carried out to examine the effect of foundation on structural response. It is found that, due to terrain irregularities and arrangement of the structural system, use of single footings and link beams in one direction only is not adequate to uniformly distribute the seismic loads. It is shown that pile foundation can have a beneficial impact on behaviour by alleviating the effects of the inclined embankment and increasing the system stiffness. The enhancement of structural behaviour is further proved by the fact that the building resting on piles can meet the collapse prevention requirements even if it is analyzed under seismic loads corresponding to the highest seismic zone in Greece, with a maximum ground acceleration A=0.36g.
Keywords: precast concrete frame, diagonal braces, pile foundation, collapse prevention.



1 Introduction

Precast concrete and particularly pre-stressed precast concrete structures have a poor reputation for seismic response. This can be mainly attributed rather to the lack of relevant experimental research in comparison with monolithic R.C. structures than to their poor performance against earthquakes, though in some cases poor behaviour during recent earthquakes has been identified [1, 2]. Furthermore, there is a common sentiment among the engineering community that the low energy absorption capacity of pre-stressed concrete structures results in higher displacement response, although severe damage or collapse does not usually occur.

In precast concrete technique, the design aim is to satisfy a 'monolithic reinforced concrete emulation' requirement [3, 4]. According to this design philosophy, a precast framed structure whose connections have sufficient strength to remain elastic under design seismic loads, with plastic hinges forming elsewhere in the structure should perform in an identical fashion to an equivalent reinforced concrete structure [5]. Various experimental studies have shown that the placement of in-situ concrete in precast frame connections can significantly improve seismic response. Restrepo *et al.* [6] tested various connection configurations typical of New Zealand construction practice and found that most of them were capable of providing reinforced concrete emulation even in the cases that plastic hinges formed in the connection region. This approach is widely accepted in New Zealand, and has recently gained favour in Japan [7].

An alternative approach has been developed in the US [8] that takes advantage of the potential efficiencies of precast construction. Unlike current building codes, the proposed recommendations aim at the construction of precast structures in which ductility is deliberately intended to occur in the connections. Hence, capacity design measures are taken to ensure that structural members will remain essentially undamaged during the design level of ground shaking and the inelastic action will be concentrated in the connection region. The main feature of the introduced connection details is the utilization of unbonded pre-stressing. Furthermore, drift-controlled seismic design than the more traditional force-based design approach is adopted.

In Greece, very few large-scale projects involving precast concrete construction have gone through and mainly involve low-rise industrial structures. This paper presents the most important analysis and design issues of the new repair and maintenance facility complex built for the Greek Railway Organization (GRO) at Thriassio Pedio, a part of the greater metropolitan area of Athens. Two single-storey buildings are the principal components of the complex. Precast concrete frames with hinged beam-column connections were used in both directions of each building due to their simple construction process and the associated cost effectiveness compared to conventional reinforced concrete design.

2 Description of the structural system

The facility includes 45,670m^2 of industrial space and 12,855m^2 of commercial office space. This project has been constructed by EDRASIS - C. PSALLIDAS

S.A. The complex comprises two building structures, called hereafter 'Building 1' and 'Building 2', which have plan dimensions of 215.6m by 165.8m and 143.1m by 133.1m, respectively. Apart from the predominant industrial occupancy, Building 1 also encompasses two- and three-storey office buildings located at its north and east sides. In Building 2, office occupancy comprises single- and two-storey buildings running along the north and the south sides of the structure. Figure 1 shows a typical plan view of Building 1.

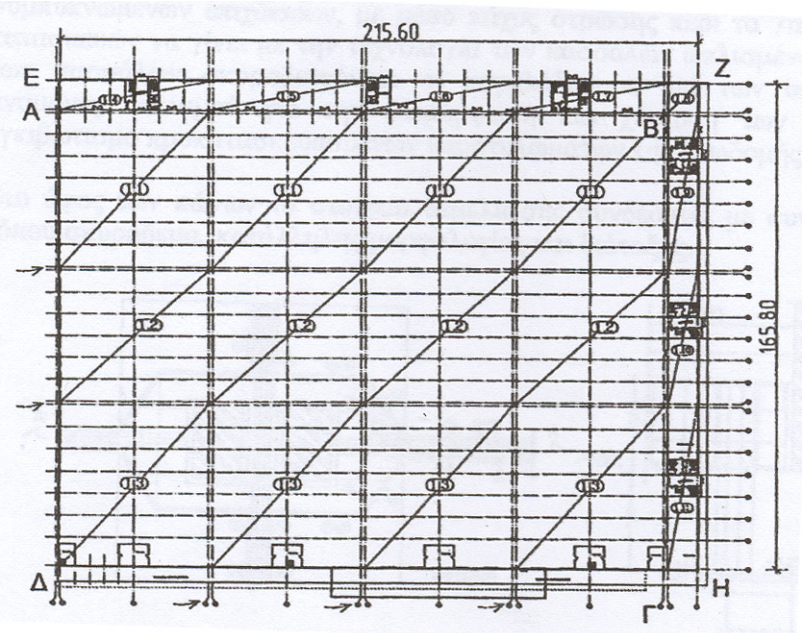


Figure 1: Typical plan view of building 1.

A thorough geotechnical study of the site showed that subsoil consists of a consolidated fill with an average thickness of 7m. Hence, a decision in favour of a pile foundation was made with pile caps interconnected through link beams made of cast in-situ concrete.

Each building is separated into several statically independent substructures. Seismic joints of sufficient width are provided between the adjacent modules, which are indicated by the dashed lines in fig. 1. Two-bay frames are used for the industrial buildings, placed parallel to each other at 7.5m spaces. Each module includes seven to nine frames. As shown in fig. 2, a constant bay width of 25m is used for the frames of Building 1, while in Building 2 two different bay width configurations are employed, either 25m and 17.5m or 17.5m and 15m.

The frame girders consist of pre-stressed precast beams, which have an I-shape of varying height (fig. 3). Three different span lengths of 24.70m, 17.20m and 14.70m are employed depending upon the location of the girder within the structure. The girders are connected to the column tops through two shear studs with a diameter of 36mm and yield strength of 500MPa.

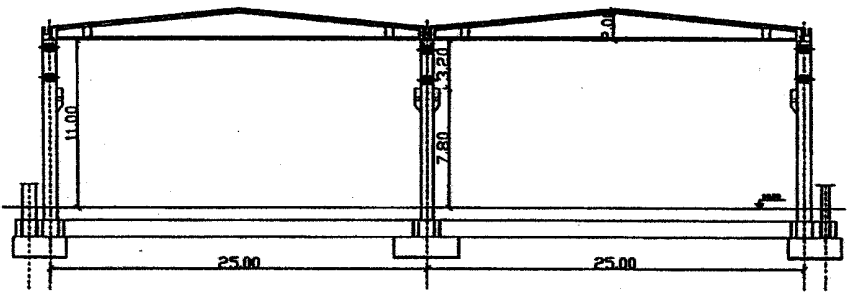


Figure 2: Two-bay frame section of building 1.

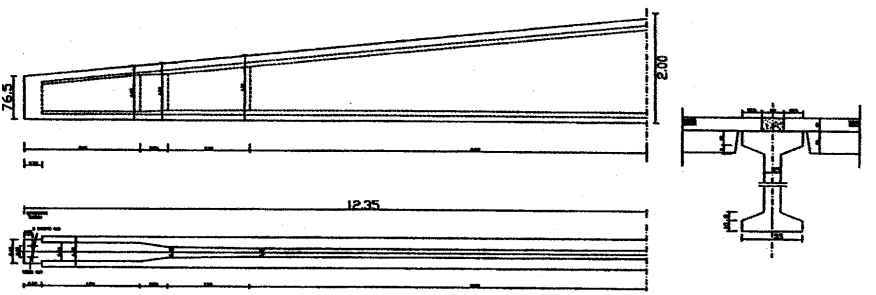


Figure 3: Geometry of pre-stressed precast girders.

The precast columns are 11m-high in Building 1 and 12.1m-high in Building 2. They have a rectangular cross section with overall dimensions 60cm by 90cm. Gutter type precast beams are used to connect the frames in the transverse direction. Floor construction consists of precast double-tee plates spanning between the frames.

The office buildings have a rectangular plan. The precast columns are placed in two parallel series at 7.5m apart. The distance between the two series is 10m. The floors consist of L-shape precast beams running around the perimeter of the buildings. Simply supported double-tee plate elements span in the transverse direction of each building. A 10-cm thick cast-in-situ concrete layer acts as topping, while it also provides membrane action, securing that the plate elements behave as diaphragms. In addition to the precast columns, the lateral force resisting system also comprises cast-in-situ reinforced concrete walls. The walls are appropriately situated and have plan dimensions of 7.5m by 10m.

3 Response of the industrial buildings

3.1 Specified design loads and material properties

Due to the importance of the facility, a 25% increase of the code specified maximum ground acceleration (A=0.16g) corresponding to seismic zone II was assumed [9]. The concrete types C20/25, C25/30 and C30/37 were used for the

cast-in-situ reinforced concrete members, precast columns, beams and plates, and pre-stressed precast girders, respectively. The steel properties were S500s, Fe 360 and $f_{PO.1K}/f_{PtK}$=1700/1900MPa for the mild steel, structural steel and prestressed steel, respectively. Other code specified parameters were: (a) importance category Σ3 (γ_I=1.15), (b) behaviour factor q=1.875 [5], (c) foundation coefficient θ=1, and (d) seismic load factor ψ_2=0.3.

3.2 Structural model and analysis results

Three different structural models were developed, corresponding to the three different frame configurations, with bay widths of 25m, 17.5m and 15m, respectively. Surface finite elements were employed to model the plates of the roof, while the ribs were simulated with linear finite elements. Columns, transverse beams, crane beams and girders were modelled using linear elements as well. Girder-to-column connections were also included in the structural model. Linear elements were utilized to model the shear studs in order to investigate the stress level on the connection components [9]. Due to the high rigidity resulting from the encasement of the columns into the pile caps, the column bases were assumed to be fixed in the development of the three-dimensional model of the structure.

Two sets of spectral analysis – dynamic and simplified – were carried out based on the current version of the Greek Seismic Code [10] requirements. A summary of the structural response of the model is given in table 1.

Table 1: Structural response of the precast frame without diagonal steel braces.

Mode shapes	
Mode	Eigenperiod (s)
1	1.352
2	0.986
3	0.983
Displacements	
Direction	Displacement (mm)
x (parallel to the girders)	66
y (perpendicular to the girders)	112

It should be noted that the first mode corresponds to lateral deformation in the transverse direction of the structure, i.e., perpendicular to the girders, while the second and the third modes are associated with twist about the centre of elastic rotation and lateral deformation in the longitudinal direction of the frame, respectively. Based on the results shown in table 1, it can be readily deduced that there is a significant difference between the structural responses in the two orthogonal directions of the frame. The structure is very flexible in the y-direction resulting in large deformations.

In order to increase the stiffness in the transverse direction of the structure, diagonal steel braces were installed interconnecting the upper parts of the

columns as shown in fig. 4. The modified frame was reanalyzed to investigate the impact of the braces on response. Two analyses were performed to obtain a more realistic approximation of actual structural response. The braces were modelled to act as truss members (i.e., they were able to take axial forces only) for the former analysis, while they were not able to sustain compressive axial forces in the latter case (i.e., cable action was assumed). The results of these analyses are summarized in table 2.



Figure 4: Three-dimensional structural model of building 1.

Based on the analysis results given in table 2, it can be concluded that the addition of the diagonal braces significantly increases the stiffness of the structure in the transverse direction resulting in practically equal stiffnesses in both axes. Furthermore, displacement in the y-direction was reduced approximately 50%. Hence, the beneficial effect of the braces on structural response is apparent.

4 Foundation design

According to the preliminary design, gravity- and earthquake-induced forces in the frame columns may be transmitted to the supporting soil by isolated footings linked with tie beams in one direction only, as specified in clause 5.4.4.2 of the current seismic code version [10]. However, a thorough study of the local site conditions pointed towards the use of pile foundations instead of single footings. It was found that the subsoil consists of a sloping fill and, therefore, a foundation on a single level cannot be achieved. As a consequence, the distribution of seismic forces is nonuniform and several local failure modes occur at the columns, such as yielding of the longitudinal reinforcement, cracking of the compressive zone of concrete due to excessive curvature, etc. Nevertheless, behaviour can be substantially improved if a pile foundation system is adopted. The enhancement in behaviour can be mainly attributed to the fact that use of

piles increases the lateral stiffness of the system. A detailed study of the structural response for each case is presented in the following paragraphs.

Table 2: Structural response of the precast frame with diagonal steel braces.

Linear analysis			
Modes		Displacements	
Mode	Eigenperiod (s)	Direction	Displacement (mm)
1	0.983	x	66
2	0.912	y	52
3	0.783		
Nonlinear analysis			
Modes		Displacements	
Mode	Eigenperiod (s)	Direction	Displacement (mm)
1	0.984	x	66
2	0.817	y	42
3	0.727		

4.1 Foundation on isolated footings

A typical structural module of the industrial building supported on isolated footings was analyzed to examine its seismic response. The dimensions of the module were 50m by 54m. According to the geotechnical study, the supporting soil deposit comprises four layers of varying thickness. The minimum and the maximum values of the shear modulus at zero strain, G_{max}, for each layer are given in table 3. It can be easily seen that G_{max} varies significantly along the fill height. Therefore, due to the inherent uncertainties related to the actual fill behaviour, the minimum value of G_{max} corresponding to each layer was assumed.

Dynamic analysis computer program SHAKE91 [11] was used to obtain the response of the layered soil deposit subjected to a ground motion. The transverse acceleration component of the 1981 Alkyonides Earthquake was applied to the top of the rocky homogeneous half-space supporting the deposit [12]. It should be noted that soil nonlinearities are taken into account by the program through an iterative procedure. Furthermore, the linear response spectrum at the top of the deposit is shown in fig. 5 for a 5% damping ratio.

Table 3: Variation of G_{max} along the fill height.

Layer	Minimum value of G_{max} (MPa)	Minimum value of G_{max} (MPa)
layer 1	48.6	97.4
layer 2	86.5	132.0
layer 3	122.0	158.0
layer 4	156.0	179.0

Foundation flexibility was simulated by using two translational and one rotational spring at each footing location. Spring stiffnesses were calculated

based on the assumption of a rigid foundation supported on an elastic, homogeneous and isotropic soil layer of finite height, which in turn rests on a rocky boundary. Due to the scattering of the soil parameter values, the following two extreme possibilities were examined: (a) rigid footings, (b) very flexible footings [13, 14]. The spring stiffness coefficients for each case are summarized in table 4.

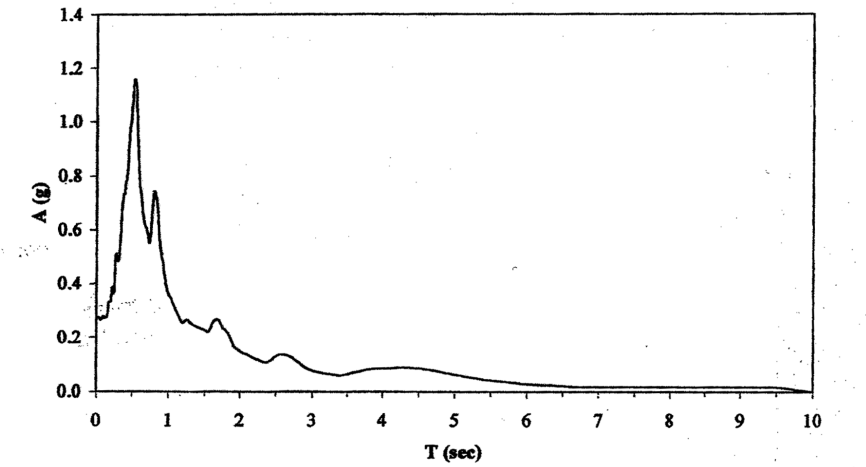


Figure 5: Response spectrum at the top of the soil deposit ($\zeta = 5\%$).

Table 4: Spring stiffness coefficients for isolated footing.

Vertical stiffness (kN/m)		Horizontal stiffness (kN/m)		Rotational stiffness (kN m/rad)	
rigid	flexible	rigid	flexible	rigid	flexible
1,861,219	1,013,199	1,263,745	717,644	5,164,332	3,001,349

In order to obtain a realistic approximation of both response and progress of damage within the structure, two sets of pushover analyses were performed. Based on the calculated elastoplastic response under seismic loads corresponding to the design earthquake (i.e., A = 0.20g), the structural members were checked in terms of strength and ductility to establish whether they satisfy the life safety performance level requirements.

For the first analysis, it was assumed that the left and the middle columns (fig. 2) are supported on rigid footings, while the right column rests on a flexible footing. Damping ratio was set equal to 2%. It was found that all columns will suffer cracking perpendicular to their longitudinal axis. In addition, the four corner columns of the module will experience significant yielding of their longitudinal reinforcement as well as concrete failure in the compressive zone due to excessive curvature. The damage on the corner columns can probably lead to collapse during a strong aftershock. A similar damage pattern resulted from

the second analysis, which involved a rigid footing at the right column and flexible footings at the middle and right columns, respectively.

4.2 Pile foundation

Soil class B (medium stiff soil) instead of C (soft soil) was assumed for the analysis of the structure on a pile foundation system. Similar to the previous case, two sets of pushover analysis were carried out. A 2% damping ratio was assumed for the first analysis, while the acceptance criteria for the check of the structural members corresponded to the life safety performance level. Limited damage occurred indicating a significant improvement in structural response compared to the response of the structure resting on isolated footings.

To verify the observed trend, a second analysis was performed. Seismic forces were calculated based on a maximum ground acceleration A=0.36g, while damping ratio was set equal to 5%. It was found that the structure can meet the collapse prevention performance level requirements with minor damage suffered by the two corner columns of the right side of the module only [15].

5 Conclusions

Analysis and design of the substructure and the superstructure of a single-storey two-bay precast concrete frame was thoroughly discussed in this paper. Due to unacceptably large drifts in the transverse direction of the structure, diagonal steel braces interconnecting the top of the columns were added. Analyses verified the beneficial effect of the braces on structural response due to the significant increase of stiffness.

A pile foundation system was implemented due to local site considerations. It was found that the structure founded on piles has superior behaviour since it satisfies not only the basic design criterion corresponding to the life safety performance level requirements but also the collapse prevention requirements when subjected to a maximum ground acceleration of 0.36g.

In summary, the seismic response of precast concrete frames can be similar if not better than the commensurate response of conventional reinforced concrete structures. Therefore, it can be a very good design option based on cost effectiveness criteria associated with standardization of the erection process.

References

[1] Hall, J.F. (editor), Northridge earthquake of January 17, 1994, Reconnaissance Report Vol. 2. Supplement to Vol. 11, *Earthquake Spectra*, 1995.

[2] Wyllie, L.A. & Filson, J.R., Armenia earthquake reconnaissance report. *Earthquake Spectra*, special supplement, EERI pub. 89-01, 1989.

[3] Tsoukantas, S.G., Seismic design of R.C precast structures, state of the art report. *Pro. of the 14th Conference on Concrete*, Kos, Greece, pp. 15-17, 2004.



[4] Tsoukantas, S.G., Main principles of the Greek code pertaining to precast R.C. structures under seismic loading, *Pro. of the 4th International Conference on Concrete under Severe Conditions (CONSEC 04)*, Seoul, Korea, 2004.
[5] ΦΕΚ 1517/27.07.1999, Greek Precast R.C. Code-Part B, Seismic Design of Precast Structures. Athens, Greece, 1999.
[6] Restrepo, J., Park, R. & Buchanan, A.H., Tests on precast concrete frame components typical of New Zealand. *Pro. of the 10th World Conference on Earthquake Engineering*, Madrid, Vol. 8, pp. 4395-4401, 1992.
[7] Priestley, M.J.N. (editor), Third meeting of the US-Japan joint technical coordinating committee on precast seismic structural systems. Report No. SSRP 92/05, Department of Structural Engineering, UCSD, La Jolla, 1992.
[8] Priestley, M.J.N., The PRESS Program – Current status and proposed plans for phase III. *PCI Journal*, **41(2)**, pp. 22-40, 1996.
[9] Spyrakos, C.C., *Finite Element Modeling in Engineering Practice*, Algor Publishing Division: Pittsburgh, P.A., 1995.
[10] Earthquake Planning and Protection Organization, Greek Seismic Code EAK2000. Athens, Greece, 2000.
[11] Idriss, I.M., User's manual for SHAKE91: a computer program for conducting equivalent linear seismic response analyses of horizontally layered soil deposits, 1993.
[12] Papazachos, B. & Papazachou, C., *The earthquakes of Greece*, Ziti editions: Thessaloniki, 1997.
[13] Spyrakos, C.C., Soil-structure interaction in practice. *Boundary Element Methods for Soil-structure Interaction*, ed. G. Oliveto & W.S. Hall, Kluwer Academic Publishers: London, pp. 235-275, 2003.
[14] Spyrakos, C.C. & Xu, C., Dynamic analysis of flexible massive strip-foundations embedded in layered soils by hybrid BEM-FEM. *Computers and Structures*, **82**, pp. 2541-2550, 2004.
[15] American Technology Council, ATC 40: Seismic Evaluation and Retrofit of Concrete Buildings, Vol. 1. California, 1996.

