THE SEISMIC BEHAVIOR OF A MULTI-STOREY R/C BUILDING THAT COLLAPSED DURING THE 1999 ATHENS EARTHQUAKE

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1 INTRODUCTION

The September 7th 1999, Athens, earthquake was the most catastrophic in the recent history of Greece, causing serious building damage and loss of human lives. Its main characteristic was not its magnitude, but the small focal depth and the proximity to the city of Athens (Gazetas, [1], Gazetas et al., [2] and Mylonakis et al., [3]).

The building examined in this study is but one of the many that collapsed. It was a R/C structure, forming one of the wings of an L-shaped structure. Located in the northern suburbs of the city, it was in the outer perimeter of the meizoseismal area. Although the design, as the analysis showed, did fulfill the code provisions effective at the time it was constructed, the building, nonetheless, collapsed.

Furthermore, the thorough investigation of the available structural, geotechnical and construction information undertaken did not provide an obvious reason for the collapse besides the fact that the structure was not able to withstand greater shaking than the one for which it was designed. Of particular interest here is the fact that the other wing of the L-shaped structure suffered only minor damage. This outlines the major importance of the dominant earthquake direction, which from the evaluation of relevant seismological data (Gazetas [1]), was indeed almost perpendicular to the weak axis of the building that failed -and consequently perpendicular to the strong axis of the other wing that remained almost intact.

All the above contribute in making this an interesting case study worthy of particular analysis and presentation. In order to examine the seismic behavior of the building in a detailed manner the following methodologies were used:

a. Analysis according to the Greek Code provisions for earthquake and R/C design that were in force at the time the building was designed.
b. Analysis according to the present day Greek Code provisions (almost identical to those of the Eurocodes, [4], [5]) using on one hand the acceleration spectra suggested by the Code and on the other hand appropriate acceleration spectra from the available earthquake records.
c. Pushover analysis based on the capacity-spectrum technique of ATC-40, [6].

The course of this study showed that compatible and in some cases complementary results were obtained by these three approaches, thus enabling the real behavior of the building to be more thoroughly understood.

2 DATA

The building under consideration was constructed in 1978 and formed one of the two wings of a four-storey L-shaped structure. The structural plan in Figure 1 shows that the two wings were perpendicular to each other, sharing the stairway and the elevator. They were statically independent, separated by a gap.

A systematic post-earthquake investigation provided as built structure information (cross-sections, reinforcement layout, etc) at the time of the earthquake. Additional information was obtained from laboratory tests of concrete cores and steel samples as well as from in-situ soil tests.

2.1 Structural System

The wing that collapsed, building 1 in Figure 1, had an area of 15.00 m x 19.50 m in plan and consisted of 6 levels including the ground floor and basement. Both the foundation depth and the storey heights were the same in the two buildings (Figure 2).

The core of the structural system consisted of 6 vertical elements whose dimensions varied with height (Table 2 shows the dimensions of the ground floor and first floor columns). The ground floor slab was a waffle type slab, 50 cm thick, seated directly on the columns, while the slabs of the other levels were ribbed, 30 cm deep and rested on the columns through beams. As it can be seen in both Figures 1 and 2, there were also cantilevered balconies in the three sides of the building.

Masonry walls, although present in all levels except the ground floor, were not as prevalent as in similar apartment buildings of that time period due to the existence of numerous openings –doors, windows, and entrances- of considerable span length.

2.2 Loads

Regarding the dead loads, both the self-weight of the structural members and the additional weights from insulation and flooring were calculated from the post-collapse investigation data. Since the building was designed strictly for housing, live loads were fairly well estimated.

2.3 Materials

To evaluate the material properties, twenty-eight concrete samples (cores) were extracted and tested in the laboratory. These tests showed that concrete of class B225 (approximately C18) was used. Similarly it was established through appropriate tests that St III (S420) steel was used for longitudinal reinforcement while St I (S220) was used for stirrups.

2.4 Soil Testing

In-situ testing showed that the bearing soil under the building was homogeneous, lacking any potentially hazardous characteristics (e.g., liquefiable sand, collapsible clay, etc).

The soil testing included boreholes in two points up to depths of 35 m approximately as well as lab tests. The results showed that the building rested on ground type A according to both the 1959 and 2000 Greek Codes and Eurocode 8. Additional evidence on the quality of the foundation and its behavior during the earthquake was provided from the inspection of the peripheral basement wall, which showed no evidence of cracking or distortion.

3 SOFTWARE

In all stages of the analysis the structural analysis program STATIK (Cubus, [7]) and its subprograms Statik-3, Cedrus-3, Fagus-3 and Pushover are used. More specifically, slabs are analyzed using the finite element program Cedrus-3 and the resulting loads are automatically transferred to the beam-column element model. The latter is analyzed through Statik-3, while the cross-section analysis is performed through Fagus-3 taking into account the exact geometry and layout of the reinforcement. This procedure provides a very good representation of the real behavior of the building up to its collapse.

4 ANALYSIS USING THE 1954 AND 1959 GREEK CODE PROVISIONS

At the time the structure was designed, the building code provisions in force were the 1954 R/C Code and the 1959 Seismic Code.

4.1 Modelling

In order for the analysis to be compatible both with the above provisions and the design practice current at the time the building was designed -one story frame model-, two independent models were used, having the same geometry and cross-sections but different nodal connections.

Specifically, for the evaluation of element forces resulting from vertical loading, model 1 was created (Figure 3a). In that model, the columns were pinned both at the top and base nodes in the direction in which frame action of the beam-column system need not be taken into account. To ensure the stability of the model, horizontal displacements of the beam-column connections were restricted. In this way beams could be analyzed as simply supported, with the ability however to carry moments only in the outer supports in each frame line.

For the design under seismic loads model 2 was used (Figure 3b). In this model the beam-column connections are fixed and additionally restrained against vertical displacement (one story frame model). This model also considers the slabs' diaphragm action.

Finally, as mentioned in section 2.4, the peripheral wall of the basement behaved as a fully-fixed base of the upper structure and thus both models in the analysis were fixed in the column base nodes of the ground floor.

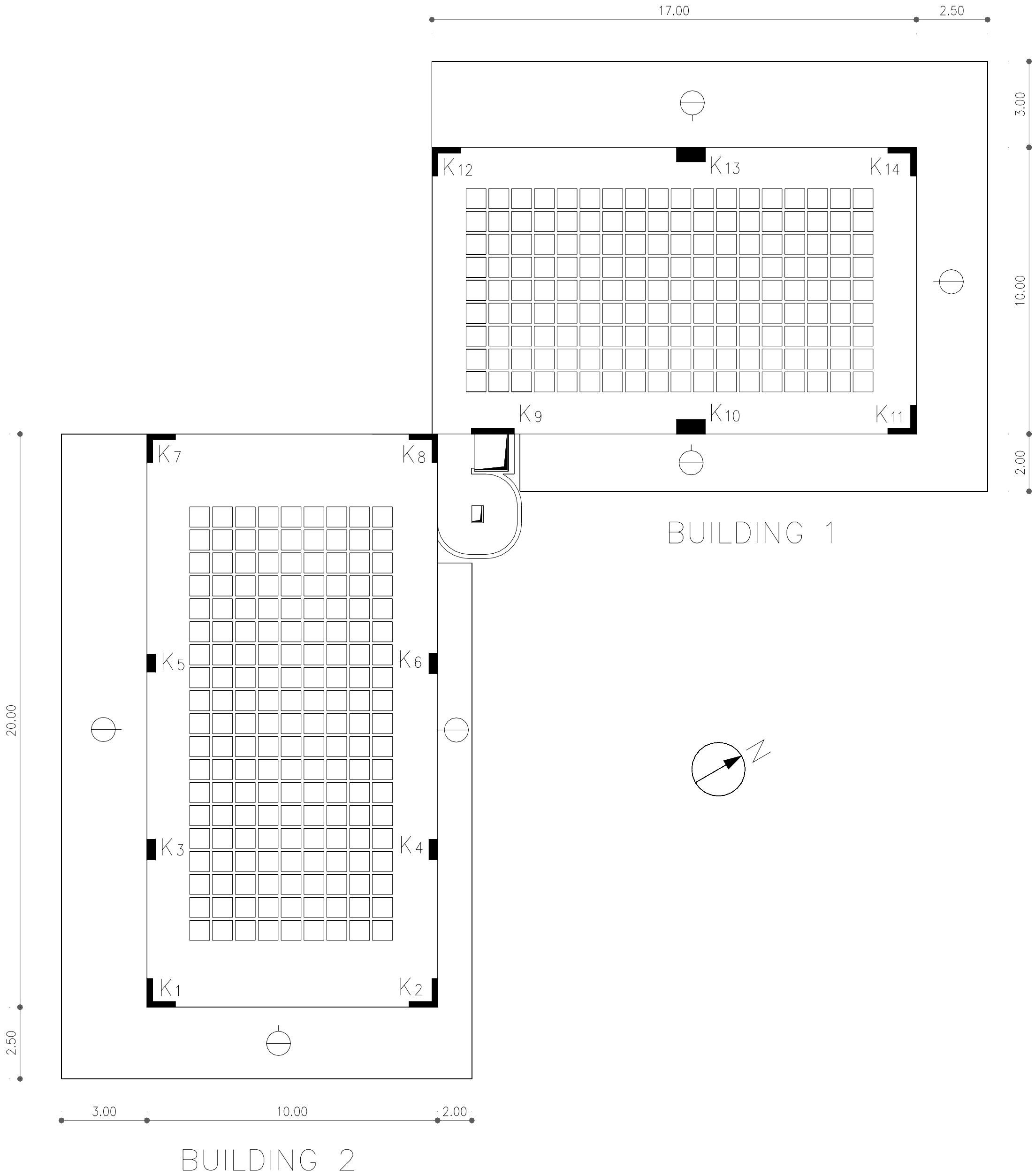

Fig. 1. Ground view

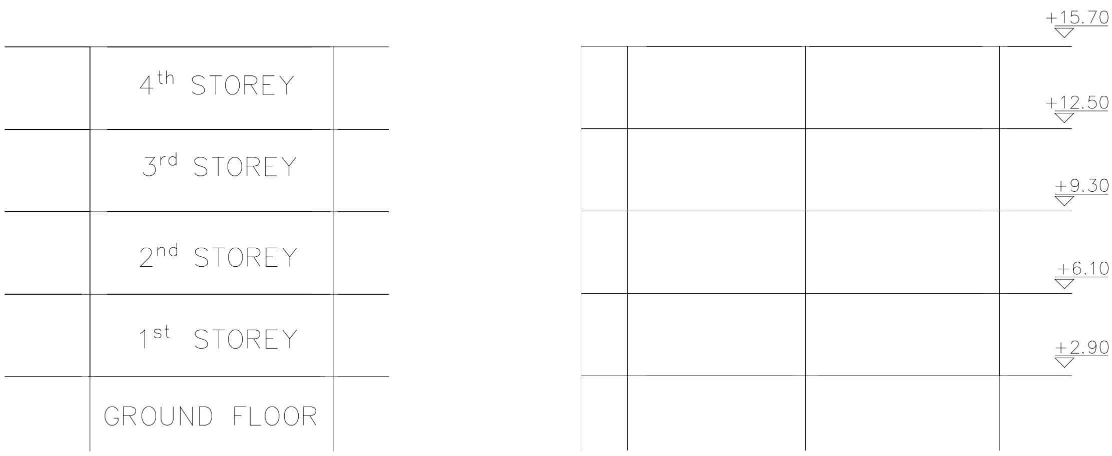

Fig. 2. Front views

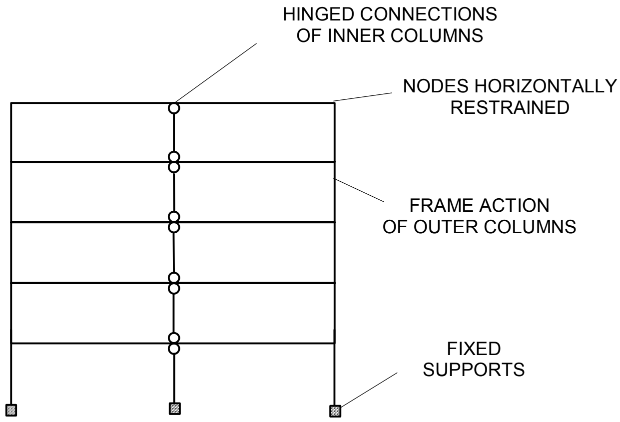

Fig. 3α. Model A for gravity load analysis according to the 1959 Code.

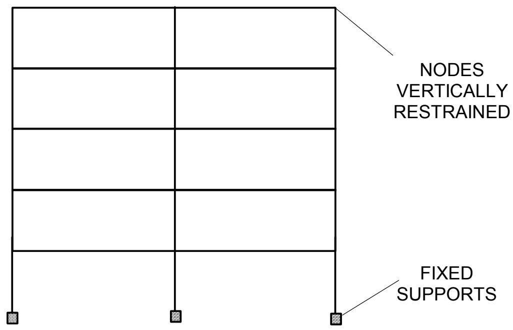

Fig. 3b. Model B for earthquake load analysis according to the 1959 Code.

4.2 Cross – Section Analysis

The cross–section analysis was practically column-oriented given the fact that column behavior was most probably the main reason for the collapse. In agreement with the 1954 R/C Code provisions, which were based on the German DIN 1045, a single safety factor of 1.75 was used for both materials, concrete and steel, for load combinations that do not include earthquake action. For load combinations that consider seismic action as well, the above safety factor was reduced by 20%, taken equal to 1.46.

In order to calculate column strength, M-N interaction diagrams from DIN 1045 (biaxial bending with axial force) were considered.

4.3 Seismic Input

An orthogonal seismic load distribution was used in both the X and Y directions. The seismic coefficient, ε, was taken equal to 0.04 according to the 1959 Seismic Code. The following load combinations were considered:

a. Gravity loads only: 1.75 (G+Q)
b. Gravity + Seismic loads in the X axis: 1.46 (G+Q) + 1.46 ε_X (G+Q)
c. Gravity + Seismic loads in the Y axis: 1.46 (G+Q) + 1.46 ε_Y (G+Q)

where:

G : dead loads
Q : live loads
ε_X : seismic coefficient in the X axis
ε_Y : seismic coefficient in the Y axis

4.4 Results

Table 2 presents the capacity indices (available member strength over the applied actions) resulting from the most critical load combinations, adjusted to the materials safety factor. Given that in every case the most critical loading involves earthquake action, a value greater than 1.46 indicates that the allowable stresses are not exceeded, while a value greater than 1.00 shows that the ultimate strength is not exceeded (theoretically no collapse occurs).

Table 2 refers to the ground and first floor columns only, where the lower values are to be found. All values are greater than 1.00 which means that no cross-section is on the ultimate strength limit. However in three cross-sections -K13 foot, K13 head and K11A head- the capacity indices are less than 1.46; this shows that the margin of safety provided by the materials safety factor is reduced so these cross-sections are the most susceptible to failure.

5 DYNAMIC ANALYSIS

The dynamic analyses were carried out in accordance with the recent (2000) Greek Codes for Earthquake and R/C design whose provisions are almost identical to the ones of the corresponding Eurocodes 8 and 2 ([4], [5]).

5.1 Modelling

The structure is analyzed as a space frame with gravity loads transmitted from the slabs as already described in section 3. The beam-column frame system is modeled with linear elements located along the centroidal axes of the members. Member properties are computed using net column lengths. Slabs are considered undeformed in their plane (diaphragm action).

The model refers to a bare frame, i.e. masonry walls in the upper levels are not considered (which would generate a soft-storey effect at the ground floor level), for the reason pointed out in section 2.1. This assumption leads to non conservative results as the increased flexibility of the frame brings the period to the decreasing branch of the spectrum. Consequently the results represent lower bounds of the actual behavior.

As already mentioned in section 2.4, the perimeter basement wall behaves as a fully-fixed base of the upper structure and thus both models in the analysis were fixed in the column base nodes of the ground floor.

The analysis is linear elastic considering small deformations (1^{st} order analysis theory), and cracked stiffness for the members. Shearing deformations are also considered.

5.2 Cross–Section Analysis

Cross–section analysis was practically column-oriented given the fact that column failure was most probably the main reason for the collapse and was carried out in agreement with the 2000 R/C Code provisions and the data provided from the site investigation.

The results are presented (Table 2) in the form of capacity indices (available member strength over the applied actions) and are derived from the envelope of all loading combinations.

Two capacity indices can be considered in seismic analysis: the first for the reduction in yielding strength considering partial safety factors for the materials, γ_s = 1.15 for steel, γ_c = 1.50 for concrete and the second for the reduction in ultimate strength, taking $\gamma_s = \gamma_c$ = 1.00.

It should be mentioned that the ultimate resistance method of the recent concrete code provisions provides different (partial) safety factors for each material, while the 1954 Code contained but a single one (1.75 for gravity loads, 1.75/1.20 = 1.46 for seismic analysis). Thus the ratio of the two capacity indices that can be defined is not constant.

To simplify the presentation of the results Table 2 summarizes the capacity indices relevant to the more meaningful ultimate strength.

5.3 Dynamic Analysis

The dynamic analysis results to eight principal modes for the structure, presented in Table 1. From the participation factors it appears that the 1^{st}, 4^{th}, and 7^{th} mode refer to the 1^{st}, 2^{nd}, and 3^{rd} modes in the Y direction, while the 2^{nd}, 5^{th}, and 8^{th} refer to the 1^{st}, 2^{nd}, and 3^{rd} modes in the X direction. Modes 3 and 6 refer to the 1^{st}, and 2^{nd} torsional modes of the model. Note that X is the “strong” and Y the “weak”, in terms of stiffness and strength, axis of the structure.

As indicated by the long natural periods and the strong participation of the higher modes the structure is indeed flexible.

As a first approximation (to be verified by the pushover analysis) a behavior factor, q =1.50, is adopted in all dynamic analyses.

The seismic combinations used are: $G + \sum \psi_2 Q_i \pm E$ where:

G : dead loads
Qi : live loads
ψ_2 : live load coefficient (in this case equal to 0.30)
E : seismic load

Table 1. Modes of vibration.

MODE	Period T (sec)	Mass Participation factor m_x (%)	Mass Participation factor m_y (%)
1	1.23	0	64
2	0.88	52	4
3	0.81	21	6
4	0.37	0	12
5	0.29	14	0
6	0.25	1	2
7	0.20	0	7
8	0.16	7	0

Table 2. Capacity Indices (C.I.) on the head (h) and at the foot (f) of columns in the state of failure (ultimate strength).

STOREY	CROSS SECTION	MEMBER		C.I. CODE 1959	C.I. CODE 2000	C.I.SYNTAGMA SPECTRUM	C.I. SEPOLIA SPECTRUM
GROUND FLOOR	K12 100/100/25	1001	f	1.65	0.24	1.06	1.10
			h	2.06	0.52	1.65	1.72
	K13 100/50	1002	f	1.84	0.55	1.49	1.51
			h	2.16	1.38	1.81	1.82
	K14 100/100/25	1003	f	1.65	0.26	1.23	1.27
			h	2.20	0.90	1.75	1.79
	K9 150/25	1004	f	1.91	0.68	1.77	1.80
			h	1.81	1.63	2.50	2.53
	K10 100/50	1005	f	2.01	0.59	1.82	1.85
			h	2.39	1.47	2.23	2.25
	K11 100/100/25	1006	f	1.55	0.31	1.32	1.35
			h	1.94	1.10	1.87	1.90
1ST STOREY	K12A 100/100/20	1101	f	1.66	0.43	1.66	1.75
			h	2.04	1.11	2.61	2.49
	K13A 60/40	1102	f	1.43	0.50	1.06	1.08
			h	1.39	0.52	1.09	1.11
	K14A 100/100/20	1103	f	1.77	0.45	1.64	1.69
			h	1.94	0.64	1.72	1.73
	K9A 150/20	1104	f	1.58	0.71	1.82	1.89
			h	1.78	1.11	2.29	2.32
	K10A 90/30	1105	f	1.81	0.64	1.55	1.60
			h	1.71	0.59	1.41	1.44
	K11A 100/100/20	1106	f	1.49	0.49	1.87	1.97
			h	1.43	0.67	1.68	1.70

5.4 Analysis based on the 2000 Greek Seismic Code spectra

In this analysis using the spectral dynamic process -which is in fact the response spectrum analysis of the Eurocode- and the design spectra suggested by the 2000 Greek Seismic Code, the following parameters were used:

Seismicity zone	: II (α = 0.16)
Importance Factor	: γ_I = 1.0
Ground Type	: A
Foundation factor	: θ = 1.00
Damping correction factor	: η = 1.00 (ζ=5%)
Behavior factor	: q = 1.50

Although the Code proposes a value of θ = 0.90, this value was considered to have been included in the behavior factor, *q* = 1.50.

5.5 Analysis using actual spectra

Although the September 7th, 1999 earthquake was recorded by fifteen accelerometers, all of them were situated at some distance from the structure under consideration. As a result the spectra used in the analyses are modified versions of the available ones from the accelerograms at Syntagma and Sepolia (based on Gazetas, [1]).

It can be noticed in Figure 5 that spectra from these motions far exceed the seismic coefficient of the 1959 Seismic Code [which is a straight line at 1.75 (q ε), where 1.75 is the conversion coefficient between the old and new codes-that is between allowable stresses and ultimate strength- *q* is the behavior factor and ε is the seismic coefficient]. On the other hand they are higher than the code spectra for low periods and lower for higher ones, the latter being of particular importance as it corresponds to the fundamental periods at the X and Y directions of the building in question.

5.6 Results

The results of the different analyses are presented in the form of capacity indices in Table 2. It can be noticed that indices are very low –most of them below 1.00- in the 2000 Seismic Code analysis. On the contrary they exceed 1.00 in the analysis based on the actual spectra. It is obvious however that even in that case indices lower than 1.15 even though they refer to ultimate strength, do indicate that the margin of safety provided by the material safety factors is certainly reduced. This is true in the cases of cross-sections K12 foot, K13A head and K13A foot. Furthermore, based on the indices related to the partial safety factors (which are not presented in the table), reduced values occur in other cross-sections as well throughout the building; namely K14 foot, K11 foot, K13B head, K13B foot, K13C head, K13C foot, K10B head, K10B foot, K10C head and K10C foot. It should be noticed that columns K10 and K13 have this problem which is in fact an indication of possible failure, in all levels above the ground floor. It is thus not surprising that the pushover analysis presented in the next section also points to these columns as the ones where failure was initiated.

6 PUSHOVER ANALYSIS

6.1 Modelling

The same model described in section 5.1 was also used for the pushover analysis.

6.2 Application

The analysis was performed through “Pushover”, a subroutine of STATIK, based on a finite step increase of the horizontal load applied to the model. The method involves step-by-step elastic analyses of the structure, which is constantly changing through the appearance of plastic hinges, until the collapse mechanism develops. The cross-section analyses are consistent with the 2000 R/C Code provisions and are based on the data provided by the post-earthquake investigation.

The loading was applied in the “weak”, in terms of stiffness, axis of the structure, which corresponds to the dominant direction of the earthquake.

6.3 Results

The above procedure indicates that the failure point is reached at a base shear V_{max} = 0.089W, W being the weight the structure, and for a maximum displacement at the top equal to 16 cm, which is approximately one percent (1%) of the building height. The failure is formed through plastic hinges at the feet of columns K10A, K13A, K14A and K11A, at the head of K10C, at the head and foot of K10D and at the head of K13D. There was also shearing failure at the feet of columns K10 and K13 of the ground floor. It should be noticed that in all cases the plastic hinge formation is related to the reduction of the cross-section of the column.

Figure 4 depicts the capacity (pushover) curve of the building as formed by the gradual increase of the horizontal load and the consequent displacement at the top. Constructing the bilinear approximation of this curve using the ATC-40 capacity spectrum technique (ATC, [6]), the elastic (yielding) point

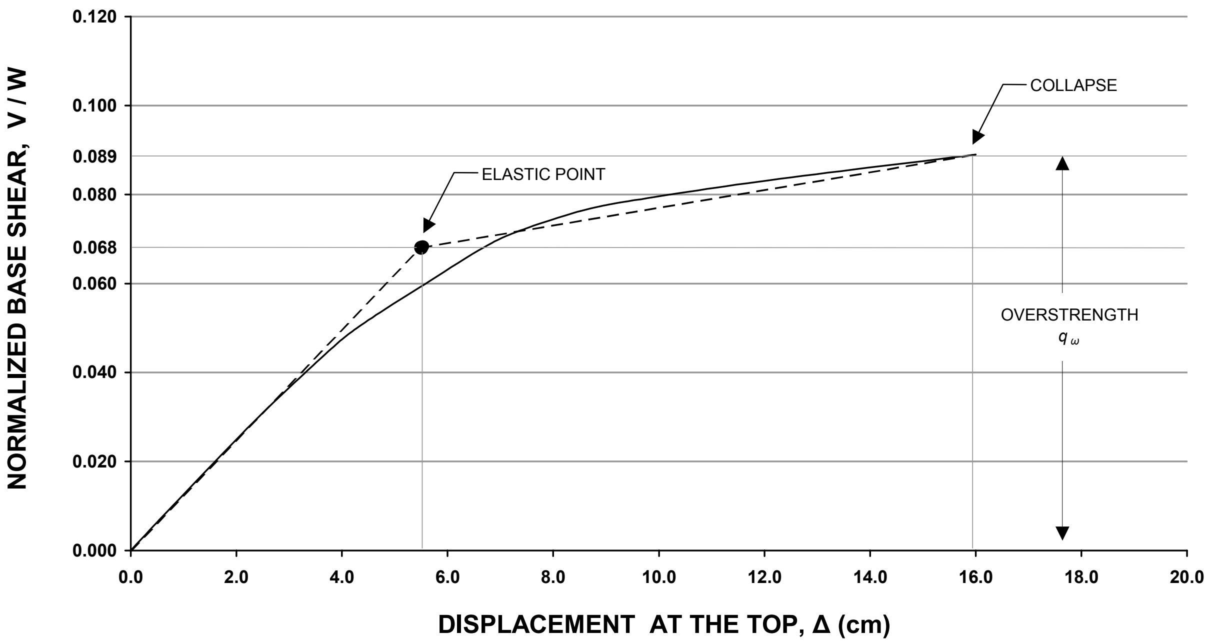

Fig. 4. Capacity (pushover) curve of the building.

is estimated at V_{el} = 0.068W. Having determined the latter, the structural overstrength factor, q_{ω}, of the building can be estimated from the relation:

$$q_{\omega} = V_{max}/V_{el} \quad (1)$$

The application of Eq. (1) leads to a value of q_{ω} =1.30.

Given that the behavior factor, q, can be defined as the product of a ductility related behavior factor, q_{μ}, and the structural overstrength factor, q_{ω}, (BSSC [8], Ghosh et al. [9]):

$$q = q_{\mu}\, q_{\omega} \quad (2)$$

it can be concluded that the behavior factor is at least 1.30. In order for the latter to have the value of 1.5 that was assumed in the previous dynamic analyses, the ductility related behavior factor for the building, q_{μ}, should have had a value of 1.15, a rather low value even for buildings of that period. Thus the assumed value of q =1.5 seems to be reasonable and most likely a lower bound of the actual behavior factor.

Following the Capacity Spectrum technique according to the ATC-40 provisions, [6], both the response spectra and the capacity curve of the building should be plotted in the spectral acceleration vs spectral displacement domain. To this end the capacity curve must be converted to a capacity spectrum and plotted against an acceleration-displacement spectrum coming from the acceleration-period spectrum.

The capacity curve is converted by dividing the horizontal load and the top displacements by the ATC-40 a_1 and C_o coefficients respectively that account for the participation of higher modes. For the structure in question, using the appropriate values of a_1 = 0.82 and C_o = 1.4 the capacity curve of Figure 4 is converted to the one of Figure 6. The value corresponding to the elastic point is 0.068/0.82 = 0.083g.

In Figure 6 the design acceleration-displacement spectrum of the 2000 Seismic Code and of a smoothened spectrum representing the seismic demand of the Athens earthquake are also shown. The smoothened spectrum is an average curve derived from Syntagma and Sepolia spectra through a smoothening process and is used in the analysis to avoid the undulations of the actual composite spectra. Evidently this process requires engineering judgment; however the authors believe that the adopted curve provides a realistic estimate of the seismic demand.

In the same figure the straight line representing the linear elastic behavior of the structure intersects the code curve at 0.175g and the smoothened spectral curve at 0.090g. Accordingly, had the building been designed using the 2000 Seismic Code, a value of q = 0.175/0.083 = 2.10 for the behavior factor would have resulted which is rather high for structures built in accordance with the requirements of the 1959 Seismic Code. On the other hand the behavior factor value corresponding to the smoothened spectrum is q = 0.090/0.083 = 1.10. As already pointed out the structure had at least a behavior factor of 1.30 (if the ductility related behavior factor is considered to be as low as 1.00). What can be reasonably concluded is that the real spectral values at the location of the structure exceeded those used in the analysis by a factor of at least 20%.

As a final remark it should be added that had the masonry walls been included in the analysis, the model periods would have decreased and the straight line representing the linear elastic behavior of the structure would have had a greater slope increasing the behavior factor values demanded.

7 CONCLUSIONS

To investigate the behavior of the building that collapsed during the 1999 Athens earthquake three types of analysis were performed:

a. Analysis according to the Greek Code provisions that were in force at the time the building was designed.
b. Analysis according to the present day Greek Code provisions (almost identical to those of the Eurocodes) using, on one hand, the acceleration spectra suggested by the code and, on the other hand, appropriate actual acceleration spectra from the available earthquake records.
c. Pushover analysis based on the capacity-spectrum technique of ATC-40.

All three approaches led to approximately the same value of seismic input imposed on the building:

- As can be seen from the elastic spectra in Figure 5 the seismic input imposed is approximately 0.10g for periods greater than 0.8 s. Having assumed a value of 1.5 for the behavior factor, elastic analyses is performed for a seismic load of 0.07g.

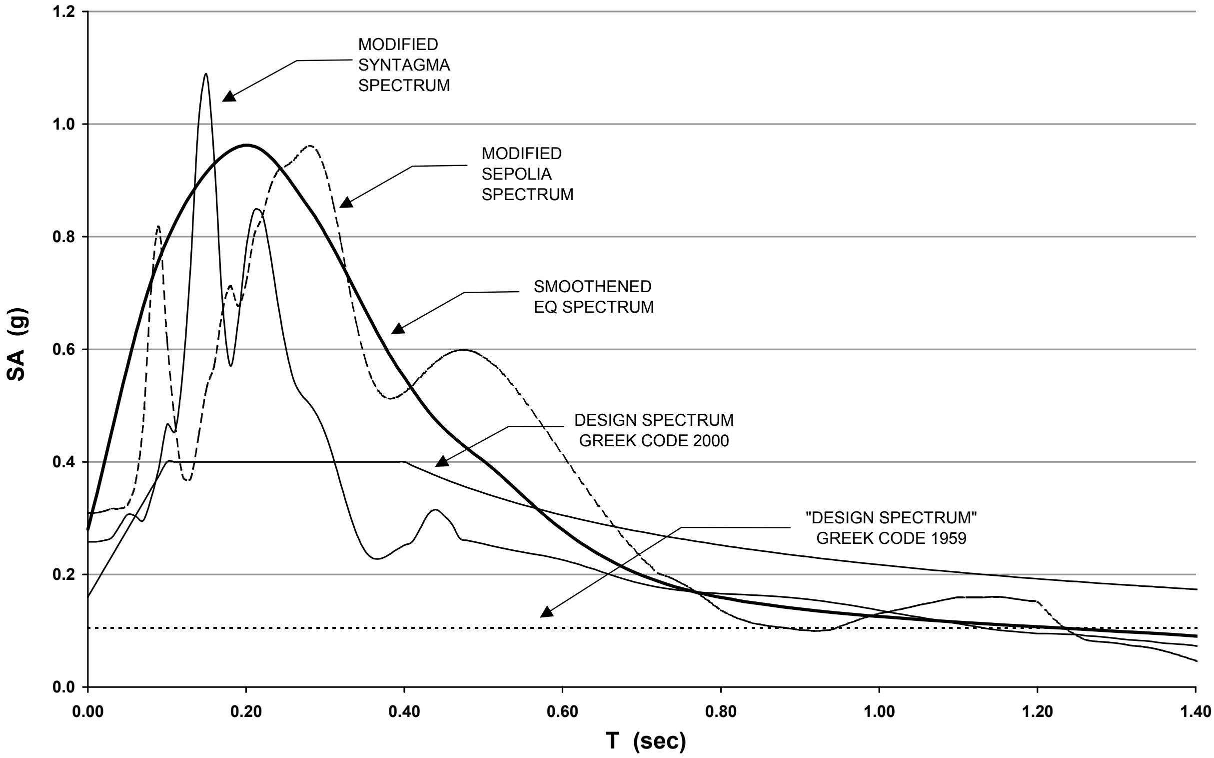

Fig. 5. Acceleration response spectra (ζ=5%).

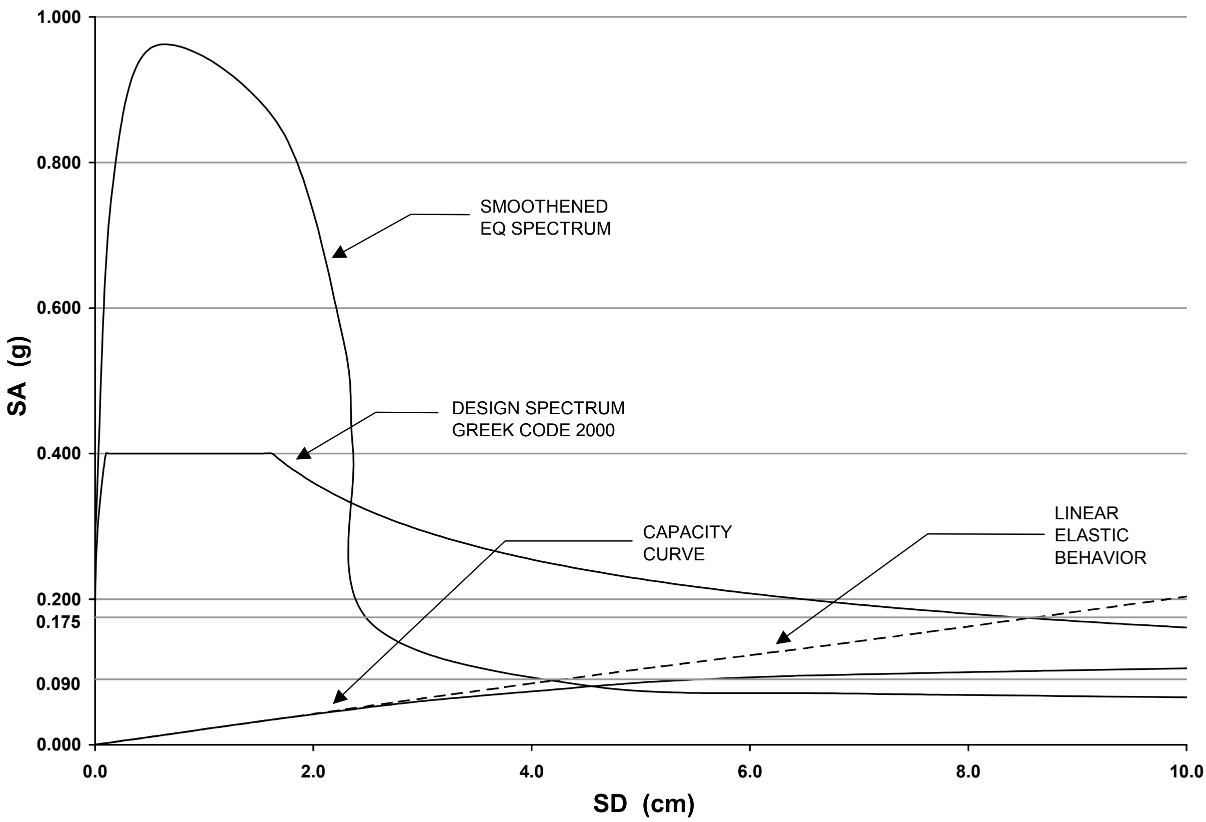

Fig. 6. Demand vs capacity spectra.

- As can be seen from the same figure, the same also applies to the analysis with the 1959 Seismic Code. The fact that the corresponding capacity indices values of Table 2 are higher in this case reflects the differing distributions of the seismic action prescribed by the two Codes.
- The Pushover analysis indicates that the structure yields at V = 0.07W (elastic point).

The corresponding capacity index values are greater than 1.00 which means that no cross-section was found on the ultimate strength limit. However in some cross-sections the capacity indices are quite low indicating that the margin of safety provided by the materials safety factor is reduced; these cross-sections are thus the most susceptible to failure. Given that the structure collapsed, the above constitute nonetheless an indication that the spectra used may have been lower than those required to initiate failure.

From the smoothened spectrum developed using the earthquake records, the demanded value of the behavior factor is 1.10. In contrast the pushover analysis indicates that the available behavior factor is at least 1.30. This provides further evidence that the actual spectral values in the location of the building were higher than those hitherto assumed in the analyses.

From this point of view it would indeed have been interesting to investigate, in the manner depicted in this study the behavior of other nearby structures. Due to the fact however that no post – earthquake investigations were undertaken in these structures the relevant in situ data were not available.

In the study presented herein the influence of the adjacent building has not been taken into account. Although separated by a sufficient gap it may have had some influence on the response of the building that collapsed.

As far as the influence of masonry walls is concerned given that they were not very widespread it is doubtful that their incorporation in the analysis would have modified the main conclusions of this study.

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REFERENCES

[1] Gazetas G., “Numerical and experimental evaluation of accelerations induced in the epicentral zone of the Athens 1999 earthquake“, 2001.
[2] Gazetas, G.; Kallou, P. V.; Psarropoulos, P. N., “Topography and soil effects in the Ms 5.9 Parnitha (Athens) earthquake: the case of Adames”, Natural Hazards, 27, pp 133-169, Oct 2002.
[3] Mylonakis, G., Voyagaki, E., Price, T. E., “Damage potential of the 1999 Athens, Greece, earthquake accelerograms”, Seventh U.S. National Conference on Earthquake Engineering Boston, (EERI electronic resource), 2002.
[4] Eurocode-8, “Design of structures for earthquake resistance”, prEN 1998-1, CEN Brussels, 2002.
[5] Eurocode-2, “Design of concrete structures”, prEN 1992-1, CEN Brussels, 2001
[6] ATC, “Seismic evaluation and retrofit of concrete buildings“, ATC 40, Vol 1, 1996.
[7] Cubus Hellas Ltd, “STATIK-3“, 1998.
[8] Building Seismic Safety Council, “NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings and Other Structures”, Washington, D. C.,1997, (2000).
[9] Ghosh S.K., Chittenden R., “2000 IBC Handbook, Structural Provisions”, ICBO, 2001.