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Performance of a 3-Storey RC Structure on Soft Soil in the M6.4 Lefkada, 2003, Greece, Earthquake

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SUMMARY

An evaluation is presented of the response of a 3-storey reinforced-concrete structure during the 2003, M_s6.4, Lefkada island earthquake. Key aspects of the event include: (1) the unusually strong levels of ground motion (PGA = 0.48g; SA_{max} = 2.2g) recorded 10 km from fault, in downtown Lefkada; (2) the surprisingly low structural damage in the area; (3) the very soft soil conditions ($V_{s,max}$ = 150m/s). The building under investigation is an elongated reinforced concrete structure of rectangular plan supported on strip footings, which suffered severe column damage in all floors in the longitudinal direction, yet minor damage in the transverse direction. Detailed spectral and time-history analyses highlight the interplay of soil, foundation and superstructure in modifying seismic demand in the two orthogonal directions of the building. It is shown that soil-structure-interaction (SSI) may affect significanly inelastic seismic response. Structural, geotechnical and seismological aspects of the earthquake are discussed.

1. INTRODUCTION

The earthquake occurred at 08:15am (05:15 GMT) August 14, 2003, with an estimated magnitude M_w=6.2 (M_s=6.4) and a seismic moment of approximately 2 x 10^{18} Nm. The source was located on several segments of a known dextral fault (approx. 14 degrees striking, 80 dipping), 130km long and 30km deep, running parallel to the west shore of the island (Fig 1). Reconnaissance reports and preliminary engineering investigations have been published, among others, by Margaris et al (2003), Gazetas (2004), and Karakostas et al (2005).

The losses induced by the earthquake were surprising low: no lives lost, less than 50 injured, one collapse, a handful RC buildings damaged beyond repair. Most of the damage was limited to non-structural elements, such as masonry and roof tiles. On the other hand, geotechnical damage was more pronounced, as extensive rockfalls occurred on the west part of the island, while quay walls in all ports suffered considerable lateral displacements.

The earthquake did not come as a surprise: there is ample historic evidence for over a dozen earthquake-induced destructions in Lefkada during the past four centuries (Papazachos & Papazachos 1998). Available information suggests that 4 to 5 events above M 6¼ are released per century. This picture seems to hold over the past century, during which five strong events took place (1914; 1948a,b; 1978; 2003). Of interest is the occurrence of some double events (e.g., 1948a, b; 1722-23; 1612-13) in the north and south parts of the island, respectively. The earthquake under investigation may be a double event, as the rupture stopped and then resurrected, several seconds later, at the island of Cephalonia (Benetatos et al 2005; Zahradnik et al 2005) 50km south of the epicenter. The stretching and segmentation of the source naturally prolongued the duration of shaking, as shown in the following sections.

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Figure 1. Rough map showing Lefkada island, fault and epicenter of main event.

2. ENGINEERING CHARACTERISTICS OF STRONG GROUND MOTION

The mainshock was recorded by seven triaxial digital accelerometers (Margaris et al 2003). One instrument was installed at the basement of a two-storey hospital building (to be called hereafter Hospital Station) in downtown Lefkada, about 10 km from fault. It provided the strongest record of the earthquake, with peak acceleration on horizontal plane 0.48g. The analyses reported herein are based on this record, which is shown Figure 2. The following are worthy of note:

- Peak ground acceleration (0.42g & 0.35g) in the two horizontal directions was unusually strong and has been exceeded in Greece only in the records of M5.8 Lefkada (1973) and M6.1 Aegion (1995) earthquakes.
- Housner Intensity, 129.2 cm, is extremely large − indicative of high spectral velocity, and has been exceeded only in the Lefkada record of 1973.
- Arias intensity, 4.08 m/s, is by far the strongest ever recorded in Greece - indicative of large duration.

The above are confirmed by the high, by Greek standards, value of bracketed duration, 15.2s, which has been exceeded only in the (much bigger) M6.7 Alkionides event of 1981. It is worth mentioning that bracketed duration is higher in the direction of smaller acceleration (N65E). The large duration is understood given the soft soil conditions, the extended seismic source, and possible directivity effects, as discussed below.

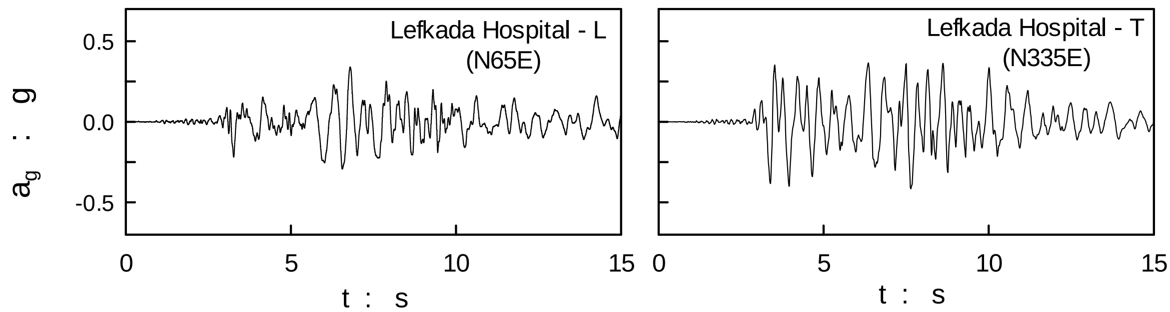

Figure 2. Time histories of horizontal acceleration recorded at Hospital station during the main event

Acceleration and velocity spectra of Hospital motion are provided in Figure 3. Values along the fault-normal direction, fault parallel direction, and envelope for all orientations are plotted. It is observed that:

- Spectral accelerations exceed the astonishing 2.2g at 0.53s and drops significantly only after about 1s. These high values can be partially attributed to seismological factors (source effects), as well as geotechnical factors (soil amplification).
- The predominant period of ground acceleration is approximately 0.55s; the record is rich in periods as high as 0.7s. These values are among the most severe ever to be recorded in Greece
- Peak spectral velocity, 1.8m/s at 0.5s, is also astonishing and provides indirect evidence for the significant sliding of rigid bodies observed in the town of Lefkada (Gazetas 2004).

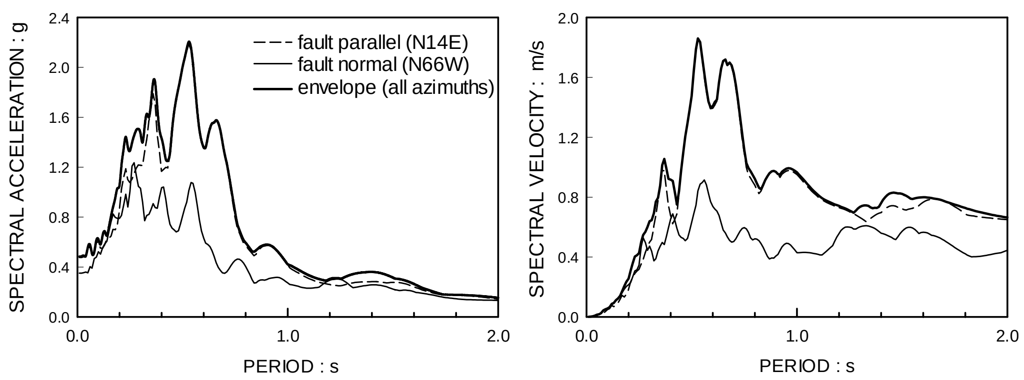

Figure 3. Response spectra of Hospital motion at different orientations; ζ = 5%

Gazetas (2004) points out that *effective peak acceleration* i.e., average spectral values in the range from 0.2s to 0.6s over 2.5

$$EPA = \int_{0.2}^{0.6} SA \, dT \tag{1}$$

attains values over 0.55g, the highest ever recorded in Greece. This value is about 25% higher than the one adopted in the current seismic code for the area (0.36g). Also important are the high values of spectral displacements (not shown), which approach 18cm at long periods. This is an important parameter which may affect the response of inelastic structures, as discussed in the following section.

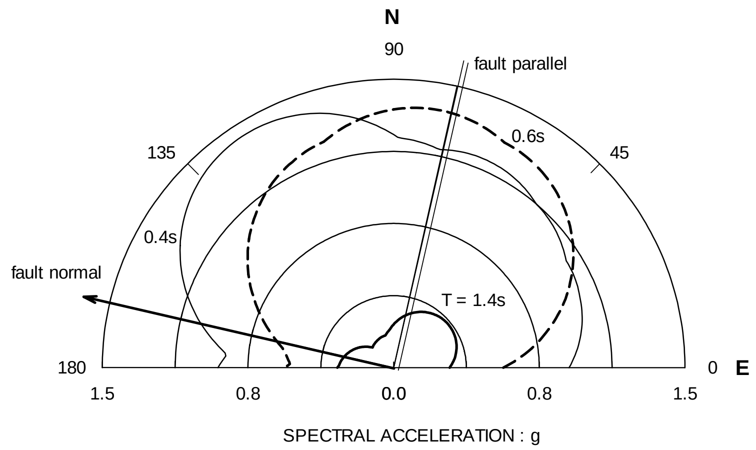

Figure 4. Polar representation of spectral accelerations for Hospital motion at different periods; ζ = 5%

Also of interest is the polar representation of SA values shown in Figure 4, plotted as function of azimuth angle for three different periods. The orientations of fault trace and the perpendicular direction are also shown. It is observed that: (1) at relatively long periods (0.6s and 1.4s), spectral values in the direction parallel to fault are significantly higher than those normal to fault. (2) In the shorter period of 0.4s, spectral acceleration in fault-normal and fault-parallel directions are of about the same magnitude. For the particular source mechanism, and given the small distance from fault, this behaviour may be attributed to *backward directivity* of the source.

3. INELASTIC RESPONSE: PRELIMINARY ASSESSMENT

Information on response of yielding single-degree-of-freedom (SDOF) structures is provided from inelastic ductility spectra shown in Figure 5. Associated parameters are:

seismic yield coefficient: $C_y = F_y / W$ (2a)
post- yielding stiffness coefficient: $\alpha = K_{py} / K$ (2b)

where F_y και W are the yield force capacity and weight of the structure, respectively; K_{py} and K are, respectively, the post-yield stiffness and elastic stiffness of an SDF structure with bilinear force-displacement characteristics. The above spectra have been evaluated for values of C_y ranging between 0.1 and 0.5. The lower bound corresponds roughly to a structure designed for low levels of seismic excitation, whereas the upper bound to a

structure designed according to the modern Greek Seismic Code (GSC) for the specific area. It is observed that ductility demand is higher in the direction parallel to fault as compared to that normal to fault.
As a preliminary step, it appears instructive to evaluate the performance of a simple structure in the light of these spectra. Given that most structures in Lefkada suffered only minor damage, to limit the inelastic demand to, say, less than 2 according to the data of Figure 4, requires

$$C_y > 0.5, \qquad T < 0.2s \tag{3}$$

The above strength requirement is severe, especially for old structures. A possible explanation is that the actual strength of traditional structures in the area (built using an ingenious combination of masonry and wood) is unusually high due to local design traditions (low weight, overstrength), possibly in the range of 4 to 5 times the design value of the code. Considering the value C_y^* = 0.12 specified by the old seismic code in conjunction with this overstrength yields C_y = (4 to 5) x 0.12 = 0.48 to 0.60, which lies in the range suggested by the spectra.

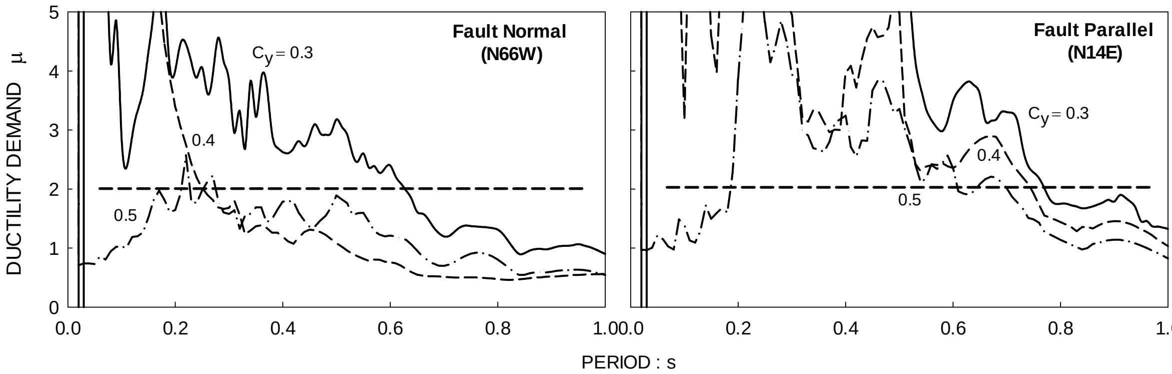

Figure 5. Ductility spectra of Lefkada Hospital motion at orientations normal and parallel to fault.

4. STRUCTURAL INFORMATION

The structure under investigation is a 3-storey reinforced concrete building in downtown Lefkada, located 200m from shore, oriented N135E (L) and N225E (T). The structure was designed in 1969, according to 1954 RC and the 1959 Seismic Codes. It was hit severely by the earthquake, especially in the lower two floors and was subsequently abandoned. A systematic post-earthquake investigation conducted by the Authors provided as-built structural information and damage information. Blueprints were also made available to the authors by the owner.
The structural system consists of three 7-column frames connected with transverse beams at the ends, which form an elongated plan, 8.80m by 31.20m (Fig 6). The column dimensions vary with height and position in plan (Figs 6, 7). The outer columns are 0.35m x 0.35m at ground floor and reduce to 0.3m x 0.3m at the upper floors. The inner columns are somewhat bigger, varying from 0.4m x 0.4m at ground floor to 0.35m x 0.35m at the upper floors. The lack of strong columns at the corners and the absence of concrete walls suggest a design focused on gravity loads – which is typical of the period. The beams have cross sections varying from 0.25m x 0.6m to 0.2m x 0.5m with inner beams being stronger than the outer. Concrete slabs are 0.15m thick in all floors with the exception of a large part of the first floor which encompasses a wooded slab. The foundation consists of three strip footings of rectangular cross section located under the main frames, built without connection beams. The outer footings have cross section of 1m x 0.5m while the middle one is 1.5m x 0.5m.
It is evident from the field observations that S220 steel was used reinforcement and C10 quality concrete. Examination of damaged columns suggests that stirrups were placed at intervals of 20cm and were well closed. This is judged as good construction by the standards of the time. According to the same standards, more weight is put in the design and construction of beams; this building is no exception to the rule.
Observed column damage was concentrated at the heads of columns on the first and second floor; severity varies from light (columns C1, C2, C21 of the 1st floor and C1, C6, C7, C8, C9, C10, C21 of the 2nd floor) to serious (C9, 1st floor and C2, C15, C19, C20, 2nd floor). Nevertheless no total member failures were observed, which can be attributed to sufficient ductility capacity of the columns.

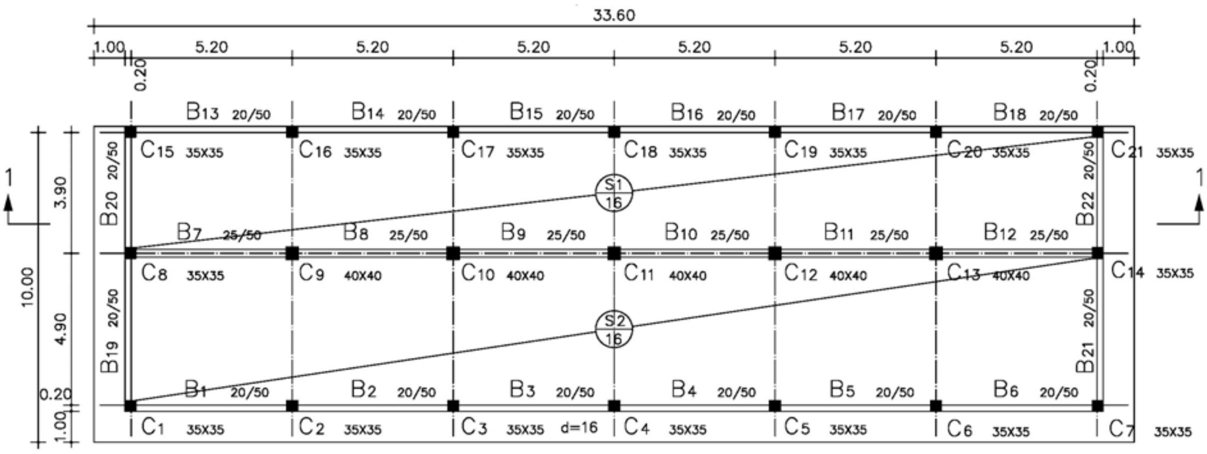

Figure 6: Plan view of typical floor

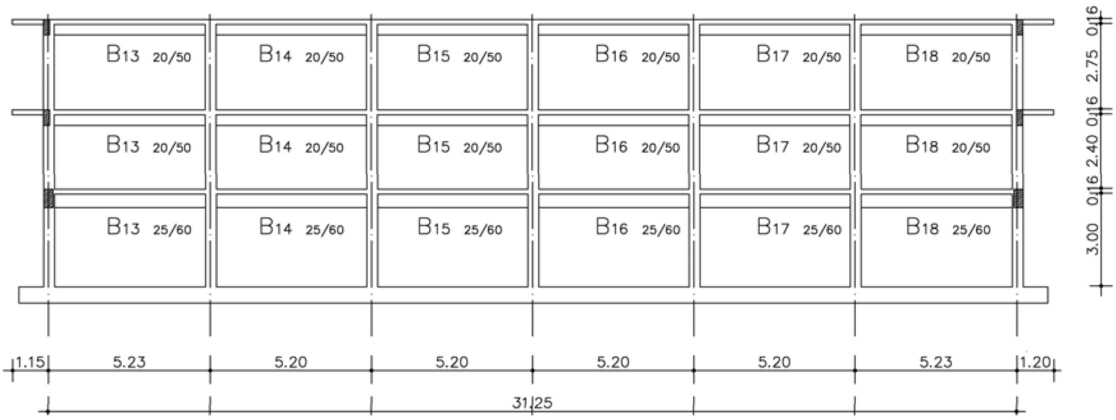

Figure 7: Section 1-1 of the building including foundation.

5. GEOTECHNICAL ENVIRONMENT

Soil conditions in the town of Lefkada are among the softest encountered in Greece. Following the earthquake, a detailed geophysical and geotechnical exploration was conducted by KEDE (2003), based on SPT and Cross Hole tests, down to a depth of 30 meters. The available data indicate the presence of a layer of stiff malr (N_{SPT} > 50) at a depth of approximately 10-12m, having shear wave velocity $V_{s,max}$ = 350m/s and thickness of over 20m. The overlying soil consists of soft alluvial deposits, primarily clayey sands and sandy silts, of $V_{s,max}$ less than 200m/s. According to the Greek Seismic Code and EC8, these formations are classified as Soil Category C. Typical cross sections at two locations are provided in Figure 8. Evidently, the differences in S-wave velocities between the soil profiles are rather small. It is possible that some of these discrepancies reflect inherent inadequacies of the exploration techniques. Our analyses of seismic soil-foundation-interaction are based on values extracted from the borehole data of Figure 8b, which is adjacent to the building.

Closed-form solutions of dynamic stiffness of spread footings have been derived by regression analysis based on finite- and boundary-element data. The validity of these expressions has been checked by different investigators over the years, and encouraging results have been obtained. Using the expressions recently published by Mylonakis et al (2006), the soil data of borehole CX-1 in Figure 4b (V_s = 100m/s, ρ_s = 1.6 Mg/m^3, ν_s = 0.33), and the foundation properties in Figure 9 i.e., foundation width $2B$ = 1m, footing embedment D = 0.5m, footing length $2L$ = 34m distributed over seven footings, the stiffness of each footing can be well approximated by:

$$K_z = 179{,}000\, KN/m\,, \qquad K_y = 196{,}000\, KN/m\,, \qquad K_x = 173{,}000\, KN/m \tag{4}$$

$$K_{rx} = 125{,}000\, KN\, m\,, \qquad K_{ry} = 735{,}000\, KN\, m \tag{5}$$

Owing to small embedment, the cross-swaying-rocking stiffness of the footings is small and can be neglected. Solutions for dashpots acting, in each degree of freedom, in parallel to the footings were likewise obtained:

$$C_z = 2500\, KN\, s/m\,, \qquad C_y = 2400\, KN\, s/m\,, \qquad C_x = 2200\, KN\, s/m \tag{6}$$

$$C_{rx} = 129\, KN\, s\, m\,, \qquad C_{ry} = 163\, KN\, s\, m \tag{7}$$

The small values of dashpot constants in the rocking modes are as expected.

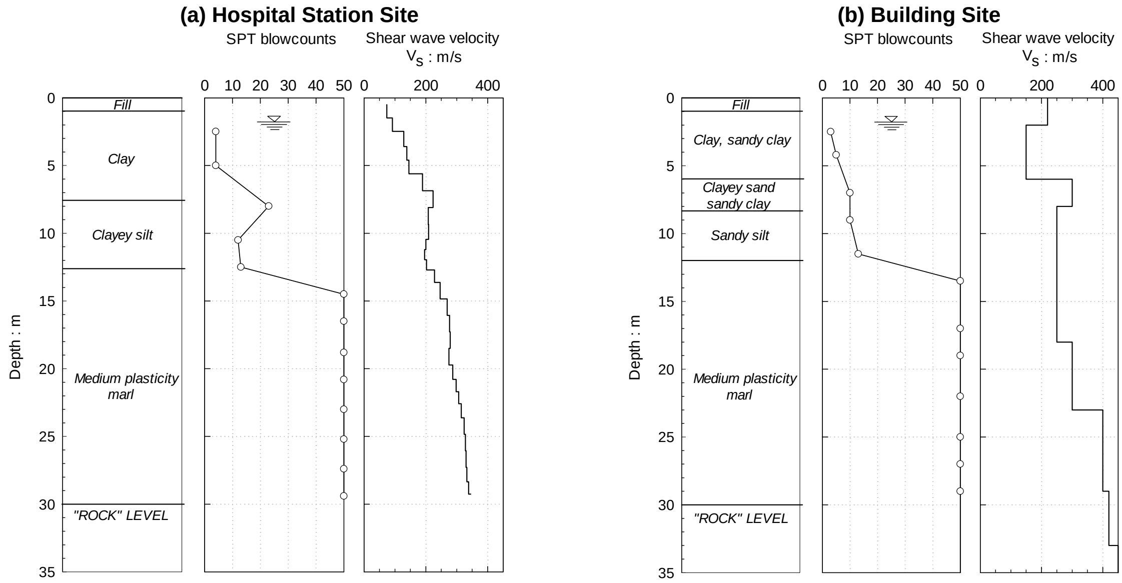

Figure 8. Soil profile and properties at Lefkada Hospital station (a) and building site (b).

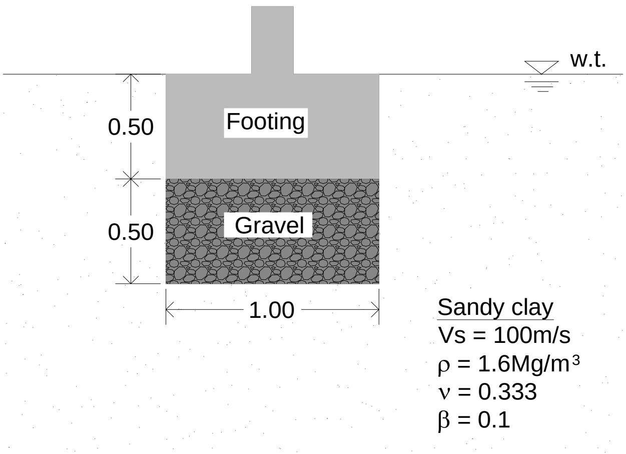

Figure 9. Foundation properties considered.

6. STRUCTURAL ANALYSIS

A variety of of analyses were performed on the structure. These include: (a) Eigenvalue and response spectrum analyses to evaluate the design according to modern codes. (b) Pushover analyses intented to investigate the probable collapse mechanism and provide information on strength and deformation capacities. (c) Non-linear time-history analyses for a more accurate evaluation of seismic demand.

6.1 Modelling

The structure is considered a space frame subjected to gravity and earthquake loading. The beam-column frame system is modeled with inelastic beam elements located along the centroidal axes of the members. With the exception of the wooden floor, slabs are considered undeformed in their own plane (diaphragms). The analysis considers cracked properties for the members according to both GSC-2000 and EC-8. Regarding supports, two alternatives are considered: (a) fixed-base conditions and (b) flexible-base conditions. The simulations are performed using the computer codes ETABS and SAP2000 which emply concentrated plasticity models.

6.2 Eigenvalue & Response Spectrum Analysis

The dynamic analysis considered the natural modes of the structure encompassing 90% of the total effective mass in each direction. For the fixed-base case, fundamental periods were estimated at 0.82s along the transverse "weak" axis, and 0.58s along the longitudinal "strong" axis of the building. For the flexible-base case, the periods were found at 0.89s and 0.6s respectively. As indicated by the long natural periods and the strong participation of the first modes, the structure is indeed very flexible at ground level.

The structure was designed according the 1959 Seismic Code, based exclusively on pseudo-dynamic considerations. The equivalent demand in modern codes would be a constant response spectrum with $R_d = 1.75$ ($q \times \varepsilon$), 1.75 being an empirical conversion factor between modern and old codes, relating provisions based on allowable stresses and ultimate strength (Anagnostopoulos et al 1987); q is the behavior factor and ε is the seismic coefficient of the 1959 code. Note, however, that the distribution of lateral loads in the two analyses is not equivalent, as the modern code follows a "first mode" distribution, whereas the former a constant distribution.

Should a value of 1.50 be adopted for the behavior factor (a reasonable assumption for old concrete structures), then $R_d \approx 2.5\ \varepsilon$. Factor ε is taken at 0.08 corresponding to an adopted value specified by the 1959 code. Accordingly, the equivalent parameters in the realm of the Greek seismic code are: Seismicity $\alpha = 0.08$, Importance factor $\gamma_I = 1$, ground type C, foundation factor $\theta = 1$, damping factor $\eta = 1$ (for ζ=5%), $q = 1.5$.

Results from these analyses – considering different support conditions – show that the structure was rather underdesigned. There is indication of failure in all corner columns, which comes as no surprise, since they were not carrying additional reinforcement (as they should had) compared to the inner columns.

6.3 Pushover Analysis

The analyses were carried out in the longitudinal the tranverse directions of the structure, both for fixed-base and for flexible-base conditions. The cross-sectional properties are consistent with the 2000 Greek RC code, based on the data provided by the post-earthquake investigation. The composite-spectrum technique recommended in ATC-40 was adopted in these analyses. Results are shown in Figure 10.

Having determined the nominal yielding strength, V_{el}, and displacement, u_y, the structural overstrength factor, q_ω, of the building can be estimated from the relation:

$$q_\omega = V_{max}/V_{el} \tag{8}$$

where V_{max} denotes maximum force sustained. Given that the overall behavior factor q can be defined as a product of a ductility-related behavior factor, q_μ, and a structural overstrength factor, q_ω, we write (BSSC, 2000)

$$q = q_\mu\, q_\omega \tag{9}$$

Following ATC-40, the capacity curve is converted by dividing the horizontal load and roof displacements by the a_I and C_o coefficients, respectively, that account for the participation of the higher modes. For the structure at hand, using the appropriate values $a_I = 0.86$ and $C_o = 1.3$, the capacity curves of Figure 10 are obtained corresponding to the L and T direction of the building.

Regarding the "strong" L direction (Fig 10a), the base shear corresponding to the elastic point is $V_{el} = 0.185W$ at a roof displacement of 1.9cm. The failure point is located at $V_{max} = 0.206W$, at a roof displacement of 17.5 cm,

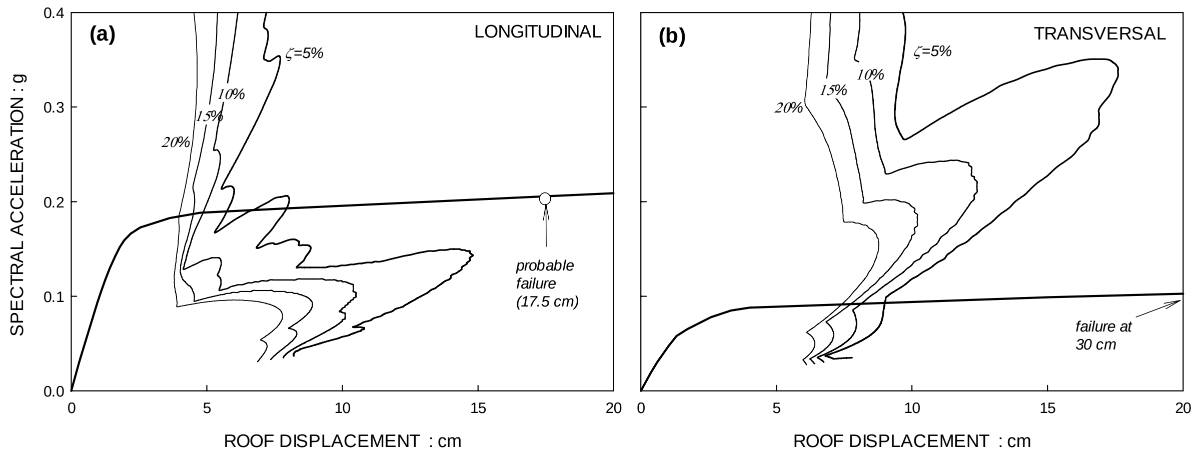

Figure 10: push-over diagrams and demand curves for the two orthogonal directions of the building.

overstrength q_{ω} = 1.10. Hinge failures start to develop at a roof displacement of approximately 4.8cm at certain columns of the 1st and 2nd storeys, including the ones that were actually damaged. Note that in most cases, formation of plastic hinges is related to reduction of the cross-section of the columns at the slab separating the 2nd and 3rd floors. As can be seen from Fig 10a, seismic demand in this direction is about 4 to 5 cm, which suggests permanent damage but no collapse.
With reference to the "weak" T direction, base shear corresponding to the elastic point is V_{el} =0.086W at a roof displacement of *1.7cm*, whereas failure point is located at V_{max} = 0.11W, at a displacement of *30cm*. This results to a system ductility capacity $\mu^{(T)} = 30/1.7 = 19$ and an overstrength q_{ω}=1.28. Plastic hinges start to appear at a displacement of 15 cm. As can be derived from Fig 10b, seismic demand in this direction is *9cm* so, obviously, at this point there is also permanent damage.
The above simplified analysis shed light on the behavior of the structure. Accepting, as a working hypothesis, the equal displacement rule, the required strength to survive the imposed motion along L direction is

$$C_y^{(L)} = \frac{SA(0.6s)}{\mu_c^{(L)}\, q_{\omega}\, g} = \frac{1.05g}{9 \times 1.1 \times g} = 0.11 \qquad (10)$$

For T direction,

$$C_y^{(T)} = \frac{SA(0.89s)}{\mu_c^{(T)}\, q_{\omega}\, g} = \frac{0.4g}{19 \times 1.3 \times g} = 0.016 \qquad (11)$$

Evidently, along T direction, the structure employs only a small fraction (15 percent or so) of its "design strength" of *0.11W*, whereas along L direction it uses over 50 percent of its design strength of *0.21W*. Naturally, this difference must be reflected in deformations, hence the more pronounced damage observed in the longitudinal direction. This feature is also indicated in direct displacement analysis using the composite spectra of Figue 10. Indeed, for conservative estimates of roof displacement demand at *5cm* and *7.5cm*, respectively, along L and T directions, the associated global ductility demands are

$$\mu^{(L)} = \frac{5\,cm}{1.9\,cm} = 2.6 \qquad (13)$$

and

$$\mu^{(T)} = \frac{7.5\,cm}{1.7\,cm} = 4.7 \qquad (14)$$

This is clearly a suprising outcome, as the less affected direction T develops almost twice the ductility demand developed in the severely hit direction L. However, comparison of these values with corresponding global *capacities* of 9 and 19 respectively, shows that the mobilization of ductility capacity in T direction is less than in L direction (29% and 25% respectively), although the difference is much smaller than the mobilization of the corresponding "design strengths". It is noteworthy that at response displacements of 5cm and 7.5cm, no member failed in the T direction - despite the existence of several plastic hinges, whereas, at the same roof displacement several members had failed in the L direction. Nevertheless, it has to be emphasized that the above pushover analysis invariably shows development of plastic hinges at the base of columns on ground floor, which was not observed in post-earthquake investigations (although no inspection was attempted below the slab).

6.4 Nonlinear Time-History Analyses

To develop additional insight on the performance of the structure, non-linear time history analyses were performed, both for fixed-base and flexible-base conditions. The member properties employed are consistent with those used in the spectral and pushover analyses. Three excitation options were considered: (1) excitation along L direction only, (2) excitation along T direction olny, (3) simultaneous excitation along L and T. To this end, the horizontal components of Lefkada Hospital recording were rotated 70^{o} clockwise to conform with the orthogonal directions of the structure. A typical result from these analyses is presented in Figure 11. It is observed that: (1) Dynamic response along L and T directions is practically uncoupled. The trend becomes stronger for inelastic conditions. (2) Peak displacements in the two directions is practically equal (about 10cm). (2) Displacement demand is similar (though not identical) to that predicted by the simplified push-over analyses (3) Elastic and inelastic displacements are comparable in all cases. (4) Response considering excitation only along the L or T direction is comparable to that for simultaneous excitation along the two orthogonal axes. (5) No significant residual offset is observed – in accordance with observations. Overall, the simulations are satisfactory.

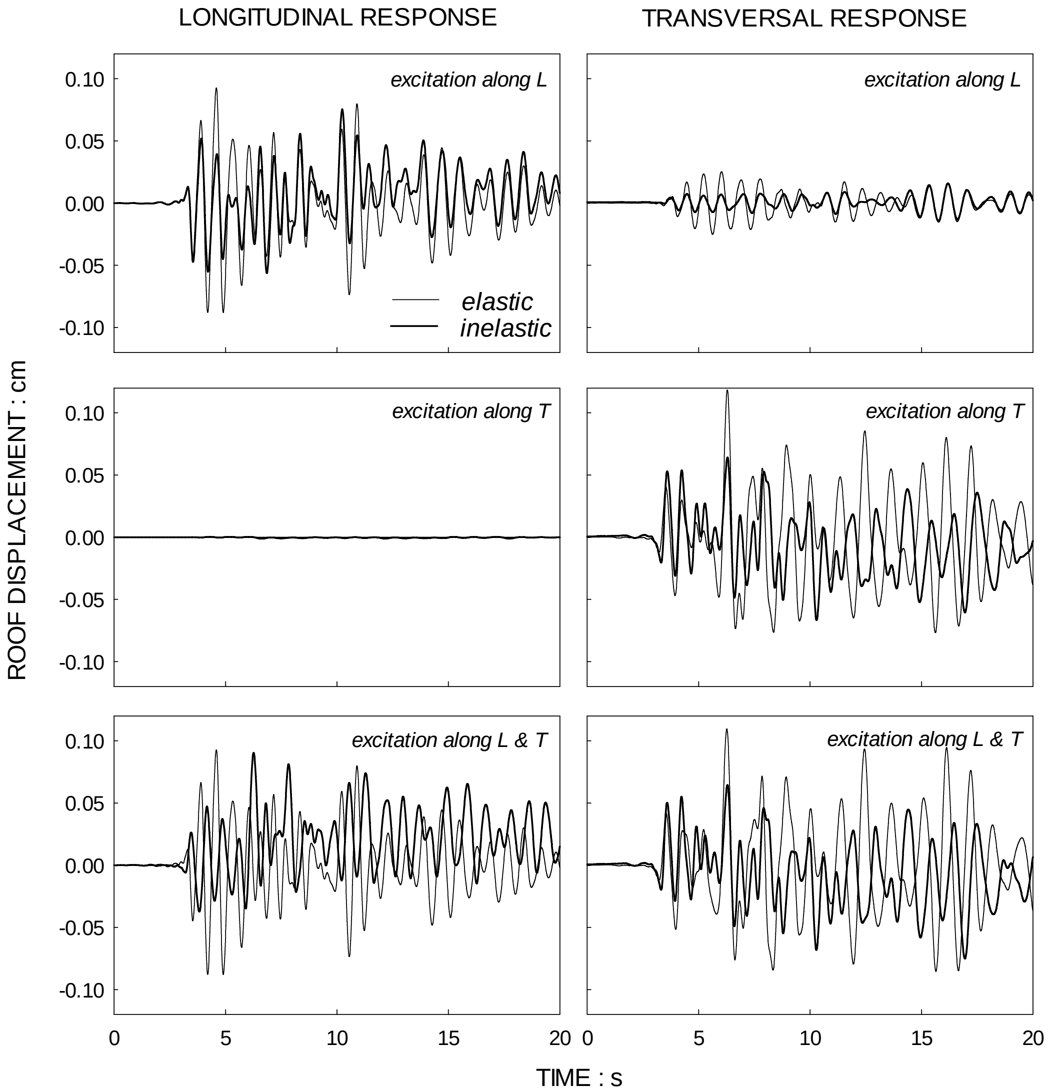

Figure 11: Roof displacements at C1 using 3D non-linear dynamic analyses for different excitations.

7. CONCLUSIONS

(1) Peak recorded ground motions were unusually strong, with effective accelerations exceeding 0.5g, and peak spectral accelerations exceeding 2.2g at a period of 0.53s.
(2) Ground motion in the town of Lefkada (10km from fault) was stronger in the fault-parallel than in the fault-normal direction. This could be attributed, among other reasons, to backward source directivity resulting from the location of the station relative to the rupture.
(3) Structural damage in the area was surprisingly low. This can be attributed to the small periods and low weights of low-rise traditional structures on the island, built using an ingeneous combination of masonry and wood. Back-calculated yielding seismic coefficients for these strucutres are estimated at 0.5 to 0.6.
(4) The structure under investigation was quite unusual, exhibiting 50% longer perior and 100% lower strength in the transverse than in the longitudinal direction. The difference in period is partly due to SSI, trigerred by the very soft soil conditions ($V_{s,max}$ = 150m/s), in conjunction with the lack of trasverse beams in the foundation.
(5) Despite the stronger displacement response and higher global ductility demand (by a factor of 2) along the “weak” direction of the building, column damage developed along the “strong” longitudinal

direction. This counterintuitive behavior is attributed to the high ductility capacity (of about 20 or so) available in the transverse direction.

(6) Simplified push-over analyses provided good estimates of building response, in reasonable agreement with more rigorous non-linear dynamic analyses. However, details of the damage were not reproduced, as analysis showed damage in locations that remained unaffected, such as the base of columns at ground floor.

(7) Non-linear dynamic analyses showed that peak roof displacements in the two directions were approximately equal (about 10cm) – despite the differences in structural properties and seismic input

8. ACKNOWLEDGMENTS

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