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Correlation of observed seismic performance of a typical Greek R.C. structure with numerical predictions

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ABSTRACT: The seismic response of typical R.C. structures built in various parts of Greece is studied numerically. A number of such typical buildings suffered, during the Kalamata 1986 earthquake, extensive damage, concentrated mainly in the ground floor. Together with the observed damage, the response of one of these structures is examined here, under lateral static or seismic loads, using either elastic or inelastic numerical analysis techniques. In order to develop a rationale for the observed failure of the ground floor R.C. members, the maximum stress levels of these elements, derived from the performed analyses, are compared with their strength properties. The type of failure implied from this comparison agrees well with the observed flexural or shear damage patterns.

1 INTRODUCTION

On September 13th, 1986 a strong earthquake of Magnitude Ms=6.2 and its strongest aftershock two days later of Magnitude Ms=5.4, subjected the city of Kalamata in the South-West of Greece to significant ground motions that resulted in the collapse of six R.C. multistory buildings and the heavy damage of a large number of R.C. multistory structures.

Extensive structural damage was sustained by a group of four identical 4-story R.C. buildings, located in the outskirts of the city of Kalamata; they belong to a category of typical multistory R.C. structures built in many parts of Greece as part of a low budget housing program supported by the Greek Government. All four buildings were the same and all of them exhibited the same kind of structural damage, namely failure of a number of ground floor columns.These typical structures have the same geometry in plan, but the number of stories varies from 4 to 6 stories depending on the housing scheme. They are designed according to current code provisions with the seismic coefficient varying according to the seismicity of the specific location. They are configured with a relatively flexible ground floor, used for parking, accompanied by an abrupt increase of their lateral stiffness at the upper floors, due to the presence of well built masonry infills at these levels. The seismic response of such a typical building is examined in this paper, named in what follows the "Kalamata building", that had just been newly occupied a few months prior to the earthquake.

2 SPECIFIC STRUCTURAL DETAILS

The four-story "Kalamata building" is a frame-shear wall reinforced-concrete structure 260 m^2 in plan (fig.1). It is formed in plan by three wings and it is founded on alluvium deposits. The foundation, located 3.0 m below the ground level, consists of separate column footings tied together with reinforced-concrete beams 300 mm wide x 500 mm deep, while a perimetric shear wall 200 mm thick is built from the foundation to the ground floor level. There are no masonry infills at the ground level, whereas the upper three stories have well built masonry infills for the apartment partitions. Reinforced-concrete slabs 15 mm thick, supported by reinforced-concrete beams 200 mm wide x 550 mm deep, carry the gravity loads. The lateral load resisting system consists of seven frames in the longitudinal direction (X) and of six frames in the transverse direction (Y, fig.1). There are four, 200 mm thick, shear walls in the structure; three of them, namely SW1,2,3, (1800mm wide) are oriented in the transverse direction, while one 1600mm wide is oriented in the longitudinal direction (SW3). The cross-sectional size of the columns decreases progressively from the lower to the upper stories. Arches are formed at the top of the perimeter columns at the ground floor level for aesthetic purposes, thus resulting in a decrease of the free structural height of these columns (fig.2).

2.1 Relevant Site information and Brief damage description

From an investigation that was carried out three days after the time of the strongest aftershock the following observations were made with regard to the structural damage of the Kalamata building :

The sustained damage is concentrated mainly at the ground floor columns and shear walls. Some

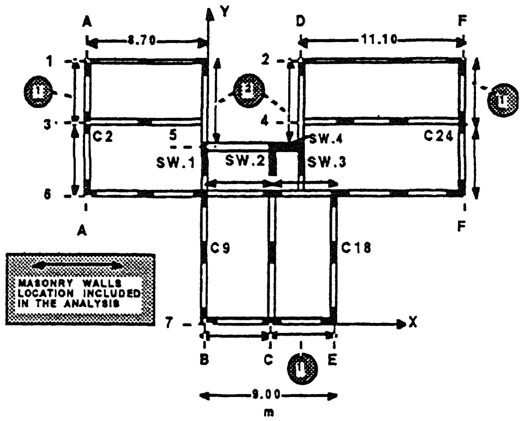


Figure 1. Plan view of the Kalamata building

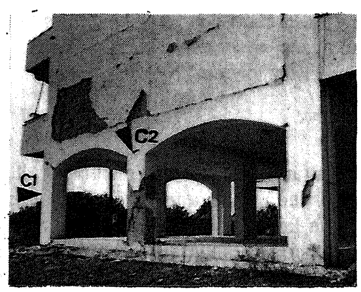
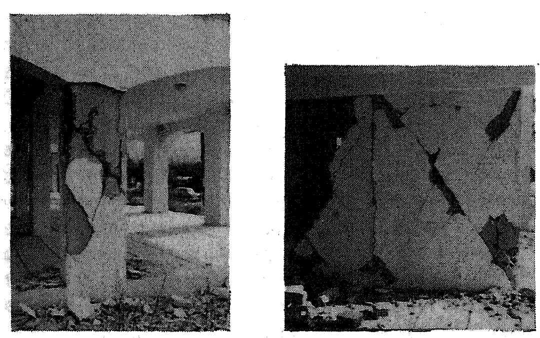


Figure 2. Damage of the ground floor structural elements and of the first story masonry infills (view from column C2 see also fig.1).

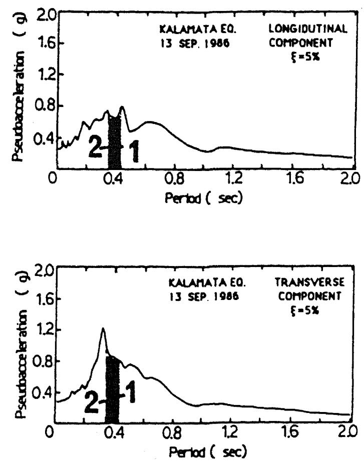


a). b).

Figure 3. Shear type failures at the ground floor level of column C24 (a) and shear wall SW3 (b).

damage is also observed at the first floor masonry infills (fig.2).

- The ground floor columns exhibited either flexural or shear type of failure. Four of these ground floor columns C2,C9,C18,C24, located at the perimeter of the building, exhibited severe diagonal-type shear failure, typical of short column failure pattern. Figure 3a



a.

b.

Figure 4. Pseudoacceleration response spectrum for the Kalamata main event.

depicts this type of damage for column C24 at the ground floor level.

- The shear wall elements also exhibited severe shear type failure, as shown in figure 3b.

- Damage was also observed at the external masonry infill panels, located at the first floor level. This damage was in the form of either limited diagonal cracks or separation of the masonry infills from the surrounding R.C. frame accompanied by spalling of the covering mortar layer (fig.2).

3 GROUND MOTION INFORMATION

Ground motion recordings for the Kalamata 1986 main event were obtained by an instrument belonging to the Institute of Engineering Seismology and Earthquake Engineering (ITSAK), which was placed at the basement of the prefecture seven-story R.C. office building [ref.1], located approximately 3km West from the Kalamata building. The peak acceleration values were 0.24g and 0.27g for the longitudinal and the transverse component, respectively. The two horizontal pseudoacceleration response spectral curves for 5% damping are shown in figures 4a,b together with the fundamental period range of the studied building, as will be outlined below. The strongest part of the horizontal ground motion is in the frequency range from 1.20 Hz to 6.6 Hz that includes the building's fundamental frequency range. This fact is indicative of the severity of the ground motion for the examined building, despite its short duration.

Despite the distance between the ground motion recording site and the Kalamata building, the numerical analyses were performed employing these strong motion records without any alterations or further treatment. This supposition of no modifications to the recorded ground motionwas supported by the following. As shown from a geotechnical survey, the soil formations that lie beneath the city of Kalamata present no remarkable differences in the East-West direction (Kalamata Building to Recording Site) with respect to the soil layer profiles and the corresponding shear wave velocities (fig.5). Moreover, as discussed by Gazetas (1989), the main event generating mechanism also supports this supposition. A more precise estimation of the ground motion near the Kalamata building could have been obtained from the available ground motion recordings and the soil layer properties employing the appropriate numerical models for predicting the surface motion variation with the distance . However, as also discussed by Gazetas (1989), such estimate of the surface ground motion attenuation did not lead to a successful conclusion. Consequently, as already mentioned, in the subsequent analyses the available ground motion recorded at a distance of 3km away from the building will be made use of.

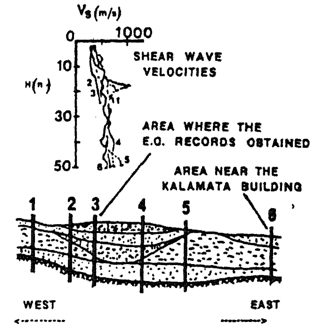


Figure 5. Soil and shear wave velocities profiles from cross-hole measurements (reproduced from ref. 2).

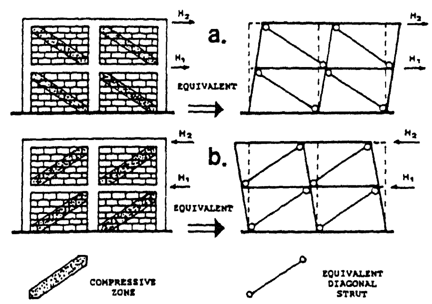


Figure 6. Configurations with the diagonal strut analogy.

4 NUMERICAL ANALYSES OF STRUCTURAL RESPONSE

4.1 Elastic analyses assumptions

The elastic analyses were performed using existing computer code [ref. 3]. The objectives of the elastic analyses were (i) to compare the structural member forces under the code horizontal static loads with the maximum member forces resulting from either a response spectrum or a step by step dynamic analysis; and (ii) to compare the capacities of the element strength properties with the maximum stress level computed from the dynamic analyses.

In all cases the structure was modeled as a single 3-D space frame. Any influences from soil-structure interaction were ignored and the ground floor columns and shear walls were assumed to be fixed at the ground level. However, the plastic hinges that were observed at the lower end of the ground floor columns after the Kalamata earthquake indicate the limitations of this fixed column end assumption. The uncracked cross-sectional properties were used to all the structural elements. Effective slab width of 1100 mm (b+6d) and 537.5 mm (b+2.25d) were used in calculating the moment of inertia of the "T" and "Γ" shaped beams respectively (where b=200 mm the width of the beams and d=150 mm the thickness of the slab). Each shear wall element was modeled using one panel element per story. The complex "Π" shaped elevator shaft was modeled as three separate panel elements having suitable "equivalent" properties.

4.2 Load combinations

First, a static elastic analysis was performed using the dead, live and seismic load levels imposed by the current Greek Building Code for R.C. buildings. For the seismic lateral loads the base horizontal shear is defined from the equation $H=\varepsilon \cdot I \cdot (G+P)$; where ε is the seismic coefficient taken equal to 0.08 for Seismic Zone II and soil category β, I is the importance factor taken equal to 1.00, G is the total dead load of the structure and P is the total live load of the structure. A triangular distribution of the base shear along the height of the building was assumed according to the code provisions. The extreme loading combinations of the vertical and horizontal static loads are :1)$G+P\pm H_x$ and 2)$G+P\pm H_y$. The static analysis was performed with or without the masonry infills.

Secondly, an elastic dynamic either step-by-step or response spectra analysis was also performed employing either of the two horizontal components of the ground acceleration, recorded during the Kalamata earthquake main event. These dynamic analyses, employing the acceleration records or the corresponding response spectra curves, were performed separately for each one of the two horizontal earthquake components taking into account the angle of the seismic input with respect to the structure reference axes (X,Y).

4.3 Masonry infills idealization

The computer code employed in the elastic analyses made it possilble to use bracing elements that can develop only axial forces in order to simulate the influence of the masonry infills on the structural

response according to the equivalent compressive strut analogy. Locations where these bracing elements were placed are shown in figure 1. The masonry infills, which were substituted by the diagonal strut analogy with bracing elements, can carry only compressive forces as they fail very easily at very low tensile stress levels. However, the bracing element of the computer code allows both tensile as well as compressive states of stress. This complication was confronted with the following rationale. In a multistory frame structure with rigid floor diaphragms carrying all the mass and with masonry infills that are not beyond their elastic range, the structural configuration of the problem during horizontal seismic input can be simplified as shown in figure 6. However, it can be easily shown that in the elastic range, for both the R.C. members and the masonry infills (masonry acting only in compression) configurations 6a and 6b are equivalent, assuming that the axial deformation of the vertical R.C. members are negligible. In this way, diagonal struts placed in only one diagonal direction and sustaining both tension and compression are equivalent to two cross- diagonal struts effective only in compression.
Following this technique, the influence of the masonry infills on the elastic seismic response of R.C. frames, with properties as those described above, was obtained by employing the appropriate "equivalent bracing elements". The validity of this assumption was checked by correlating the numerical results of such an elastic dynamic analysis with the corresponding experimental results from simulated earthquake tests of a two story R.C. model that had various combinations of actual diagonal braces placed on its frames. Good agreement was found between these experimental and the numerical results [ref. 4].

The modulus of elasticity for the bracing elements used in the present study was taken equal to 2500 Mpa as suggested by Manos (1988). The effective strut width was calculated using formulas given by ref. [6] and was taken equal to 0.22d and 0.17d for the wall infills coded in figure 1 with the numbers 1 and 2, respectively.

4.4 Discussion of the elastic analysis results

a. Figures 4a,b depict the variation of the fundamental period in the X and Y directions (shadowed area), as derived from the analysis with or without masonry infills (lines 2 and 1, respectively). From these figures it can be seen that the fundamental translational period in each direction, is within the range of the maximum spectral ordinates.
b. Figure 7 depicts the floor displacements for the static analysis with or without masonry infills. As can be seen, when the masonry infills are included, the floor displacements are decreased by 25% . This reduction becomes more significant when masonry infills are also placed at the ground floor. Although this does not correspond to the prototype structure, it was included in order to investigate the influence of such a possibility .

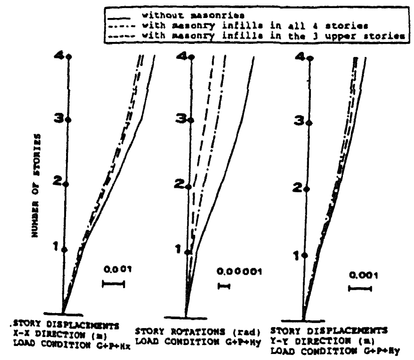


Figure 7. Story displacements from the static elastic analyses using the code lateral loads.

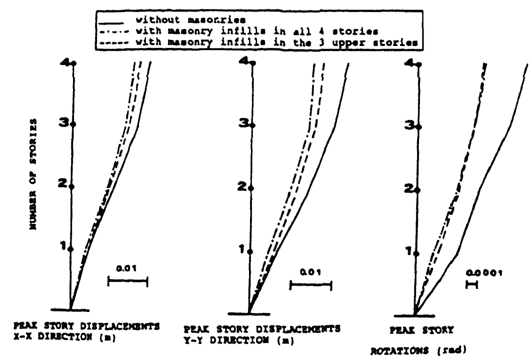
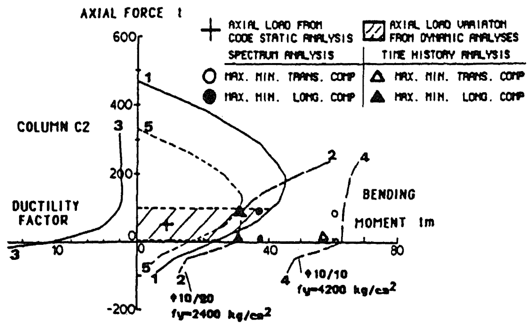


Figure 8. Peak story displacements from the dynamic elastic analyses.

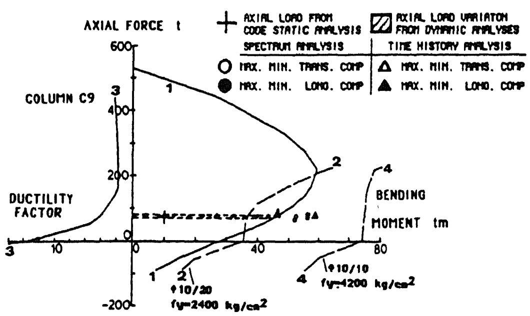
c. Figure 8 depicts the peak floor displacements as derived from the dynamic analyses, again with or without masonry infills. As can be seen by comparing figures 8 and 7, the dynamic displacement response based on the actual earthquake motion is approximately 5 times larger than the corresponding values resulting from the code static load analysis.
d. A significant reduction is also observed in the bending moments and shear forces of the structural elements above the ground floor when the masonry infills are included in the analyses.

4.5 Column Strength under Moment Axial and Shear State of Stress

From the presented dynamic elastic analyses results, which were derived by employing the actual ground motion, it was shown that the various structural R.C. elements were subjected to large displacement and stress levels, more than five times those specified by the code. Moreover, the well built masonry infills at the upper stories of the structure, although they decreased the stress level of the R.C. elements above the ground floor, they overstressed those on the ground floor in a



a). Column C2



b). Column C9

Figure 9. Moment-Axial interaction diagrams for columns C2 (a) and C9 (b) exhibited severe shear failure.

typical soft story manner, thus partly leading to the observed damage.

Apart from the above general observations, in order to explain the development of the specific damage patterns at the ground floor columns the following procedure was used. The shear or the flexural strength capacities of the various columns are compared with the maximum internal forces computed from the dynamic elastic analyses; if exceeded failure is indicated. Such type of correlations have also been used by other researchers in order to explain observed damage patterns of R.C. structures subjected to earthquake excitations [refs.7,8]. The flexural or shear strength capacity of the ground floor columns were obtained, based on the cross sectional data of the original design plans. In what follows the correlation for a limited number of ground floor columns with typical damage patterns is discussed. Figures 9a,b depict the moment-axial (M-N) interaction diagram of columns C2 and C9 respectively at the ground floor level.

Curve **1** in figures 9a,b corresponds to the M-N envelope based on the columns flexural strength, curve **2** corresponds to the M-N envelop based on the column's shear strength, while curve **3** is the ductility factor diagram corresponding to various axial force levels. In these figures the moment-axial load combination corresponding to the code static analysis is indicated by a cross (+)

The same figures (9a,b) also show, together with the M-N diagrams, the combination of the maximum axial and bending moment column forces obtained from the dynamic elastic analyses. The circles (o,●) in these plots represent the maximum or minimum values as obtained from a Kalamata response spectrum dynamic analysis whereas the triangles (Δ,▲) are the corresponding results based on a step by step integration using the recorded accelerograms. The shaded area in the same figures indicates the variation of the axial load (maximum- minimum) obtained from these dynamic analyses. The following summarise the main points.

a1. From the static analysis results it can be seen that the supplied strength of these R.C. members is adequate to withstand the forces developed from the lateral loads specified by the code. It must be pointed out here that these M-N diagrams were obtained without using capacity reduction factors. Even in the case of using capacity reduction factors, with values equal to 0.7 for the columns (curve 5) and 0.85 for the shear walls, the supplied strength is also superior to the demand due to the code static lateral loads, a fact shown not to be valid by the observed structural damage.

b1. It can be seen that the shear strength of the columns is smaller than the flexural strength for the above mentioned axial load variation. Moreover, the computed dynamic analyses results show a demand larger than the strength capacity, which implies a premature failure in shear rather than in flexure, a fact born out by the observed damage. The calculated flexural ductility factor for the ground floor columns is about 2 to 4, although this should not hide the above mentioned shear failure potential.

c1. The shear strength of the existing columns could have been increased by applying a better confinement through closely spaced stirrups; the corresponding values in that case are represented by curve 4 in figures 9a,b. If this proposed transverse reinforcement arrangement was followed in design then the shear strength of the columns would have been larger than the maximum stress level demands obtained from the dynamic analyses using the actual earthquake loads.

d1. For the shear walls the maximum shear stress from the dynamic analyses was approximately $0.4\sqrt{fc'}$ to $0.66\sqrt{fc'}$ indicating significant shear stress levels. For the shear wall SW.3 the maximum base shear was twice as much as the one calculated by the formula : $Q = b \cdot d \cdot (v_s+v_c) = 66.6t$ [ref. 9], where b,d are the shear wall width and depth, respectively; v_s, v_c are the horizontal steel mesh and the concrete contribution to the shear strength of the shear wall.

e1. The ratio of the maximum base shear, which is derived from the dynamic analyses, over the total weight of the structure (Qmax/W) is equal to 0.43. This value when compared with the code seismic coefficient (ε=0.08) is about five times larger. Moreover, even in the case that a reduction can be considered by dividing this value by an assumed ductility factor equal to two for the ground floor columns, the resulting ratio is still approximately 3 times larger than the code seismic coefficient. That signifies the increased demands posed on the structure by the actual earthquake excitation leading in combination with the remarks of **b1** above to the observed damage.

5. INELASTIC ANALYSIS

5.1 Inelastic analysis assumptions

The inelastic analysis was performed by the computer program DRAIN2D/85. Only those frames that are parallel to the (Y) direction are analysed (see fig.1); this is due to the significant damage observed at the ground floor columns and shear walls parallel to that direction. The analysis was based on the transverse component of the ground acceleration recorded during the Kalamata earthquake, because it was found from the elastic dynamic analysis that this component caused the maximum stress levels on the structural members. For this purpose the transverse component was assumed to act parallel to the (Y) direction. The analysis was performed with or without the masonry infills. Any soil-structure interaction was neglected and the columns were assumed to be fixed at the ground floor level, as was also done in the elastic analysis.

5.2 Inelastic analysis results

1. The maximum displacements calculated from the inelastic analysis are twice as much as the corresponding elastic dynamic analysis values. This is mainly attributed to the inelastic behaviour of the structural members; to a lesser degree this is also due to the use of the cracked stiffness geometric properties for the elements in the inelastic analysis, whereas for the elastic analysis these stiffnesses were derived from the gross-section geometric properties. The corresponding stress levels are almost three to four times larger than the corresponding maximum values resulting from the code seismic load case.

2. When masonry infills are included in the analysis the results indicate structural damage (e.g formation of plastic hinges) concentrated mainly at the R.C. columns, beams and shear walls of the ground and first floor level, and to the first floor masonry infills. When masonry infills are omitted, plastic hinges develop at almost all the structural elements from the ground to the fourth floor.

6. SUMMARY AND CONCLUSIONS

1. Despite the inelastic behaviour of the Kalamata building, the performed dynamic elastic analyses provide an insight to the seismic response of this typical 4-story building. From these analyses, which included influences from the masonry infills and employed the actual ground motion, the overstress of the ground floor vertical members becomes evident. The approach used in order to explain the observed failure of the ground floor columns, by comparing the flexural and shear strength capacities with the demand specified from the dynamic analyses resutls, yields good agreement between observed behavior and predicted damage potential.

2. The ground floor columns strength capacities are larger than those demanded by the design according to the code seismic load provisions. However, ground floor columns and shear walls stress levels obtained by considering the recorded ground acceleration are many times higher that those computed using the static lateral loads of the code, thus explaining the observed damage.

3. As was shown by the numerical analyses (either elastic or inelastic) the masonry infills at the three upper stories decrease the internal forces at the structural elements at those floor levels. This results in the ground floor columns being more distressed than those of the upper stories. Thus, the influence exerted by the masonry infills is far from insignificant and its omission leads to non-conservative results.

4. For the range of the axial load variation, as was computed from the elastic dynamic analysis, the flexural capacity of the columns failed in shear is greater than the shear capacity. Moreover the shear capacities of the ground floor column failed in shear are not adequate to withstand the forces computed from the dynamic elastic analysis, a fact born out by the observed damage.

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