

# RESPONSE OF A R/C BUILDING IN CEPHALONIA (GR) TO EARTHQUAKE EXCITATIONS DURING THE 26/01-03/02/2014 SEISMIC SEQUENCE

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### Abstract

The seismic sequence in the island of Cephalonia, Greece, in the beginning of 2014, which included two chronically close strong motions, raised the problem of determining the structural response due to the two consecutive strong seismic events. Due to the fact that the modern seismic codes do not include specifications dealing with such series of events, the majority of structures are analysed considering one design seismic event corresponding to an assumed specific occurrence probability during their structural life. The current study investigates the response of an existent r/c building in the region of Lixouri, in the island of Cephalonia, designed on the basis of the old Greek seismic code, applying the specific consecutive seismic events. By means of the results obtained, an attempt is made to test the potential ability of the finite element model for the approximation and justification of the real observed damage and, additionally, to explain the observed damage state after the second strong seismic event. For this purpose, non-linear time response analyses (NTRA) were performed using the available accelerograms. The results led to the conclusion that the observed damage state is generally verified by the results of NTRA. Finally, the results of the analyses show that the observed damage state, which is characterized by moderate damage in masonry infill walls and no damage in r/c structural members, is due primarily to the participation of the masonry infill walls in the seismic response of the building and secondarily to the foundation soil category.

Keywords: Repeated earthquakes; Seismic sequence; Seismic response; 3D R/C buildings; Seismic damage



## 1. Introduction

The seismic sequence at the beginning of 2014 in the island of Cephalonia, Greece and especially the two major events ( $M_w$ =6.1 on January 26, 2014 with recorded PGA=0.54g and  $M_w$ =6.0 on February 3, 2014 with recorded horizontal PGA=0.68g in the town of Lixouri) [1] underlined, among other things, the issue of the response of a structure to consecutive seismic excitations, with no strengthening interventions in-between. The extremely short time span between the two major events made unfeasible even minor (let alone more extensive) repair/strengthening interventions to buildings with structural damage due to the first event. The exceptionally high measured PGAs of both events which led, in some cases, to a significant level of structural damage of buildings during the first event, led to further damage later due to the following major excitation.

It is well-known that the seismic codes (e.g. [2]) prescribe as the maximum excitation level for the design of new structures the one of the "design earthquake" with a given possibility of non-exceedance (in terms of its PGA) during the lifespan of structures. With a code-prescribed PGA value of 0.36g, it is obvious that the structures in Cephalonia were subjected to higher PGA excitations twice, within a very small time span.

The present research effort aims at the investigation of the seismic response of an actual r/c building in Lixouri during the 26/01/2014 and 03/02/2014 major events, through nonlinear time-response analyses based on respective acceleration records from a) the National Observatory of Athens (NOA) strong motion network for the first event and b) EPPO-ITSAK strong motion network on the island for the second event. More specifically, using special analysis tools, the initial level of damage due to the first major excitation is firstly assessed, and then the cumulative damage due to the sequence of both events is evaluated. In order to verify the analytical prediction of damage, this prediction is later compared to the actual level of damage observed for the building under investigation.

The results show that the masonry infill walls played a major role in the almost null level of damage of the r/c structural members. Furthermore, the impact of the second strong event on the deterioration of the damage of the masonry infill walls was documented. Finally, the ability of the FE models to approximate the observed actual level of damage to a very satisfactory level was also documented.

### 2. Description of the investigated building and modeling assumptions

### 2.1 Description of the building

The r/c building analysed is located in the town of Lixouri in the island of Cephalonia, Greece and was constructed in 1985 under the Greek codes applying at that time. It is a two-storey building with basement and a loft (Fig. 1). The materials employed for the construction of the bearing system were concrete grade B225 and steel grade StIIIa, according to the categorization permitted within the framework of the corresponding codes. The height of both the ground floor and the first floor is 3.0m, while the height of the basement is 1.95m. As for the loft, which is covered by a tiled roof, the maximum height is 4.0m and the minimum 1.0m. The loft and the tiled roof, which constituted a storey addition, were constructed in 1993. The loft was made of lightweight materials and it was thus considered that it could only be introduced in the model as a permanent vertical load. In a part of the perimeter of the basement, r/c walls were constructed (Fig. 1d).

The structural type of the building is "frame system" (system of spatial r/c frames without r/c walls). The floor plans at all levels are non-regular (Fig. 1a-d) according to EN19981-1 criteria. The columns are located mainly at the perimeter of the building (Fig. 1a-d), and they have a quadrangular cross-section (with dimensions 0.45mx0.45m in all levels). The longitudinal reinforcement of columns consists of 12 bars of 20mm in the basement and the ground floor and 8 bars of 20mm in the first floor. The transverse reinforcement of columns consists of stirrups of 8mm placed every 15cm. All beams have a cross-section of 0.25/0.60m at all levels (except for beams B11 (Fig. 1a-c), which bear the staircase, with a cross-section of 0.25/1.00m). The longitudinal and transverse reinforcement of beams are shown in the constructional drawings, which were available to the authors. The slabs of the building are of two types: there are solid slabs and ribbed slabs (zoellner slabs), as shown in Fig.1a-c. The foundation of the building consists of r/c foundation beams which



have trapezoidal cross-sections (with a height of 1.0m and width of 1.0m at the base and 0.5m at the top, Fig. 1d). The building has strong masonry infill walls in the perimetrical frames and lightweight ones in the interior.



Fig. 1 – Structural design drawings of (a) the roof of the 1<sup>st</sup> floor, (b) of the roof of the ground floor, (c) of the roof of the basement and (d) of the foundation

### 2.2 Modelling assumptions

Concerning the modelling of the linear and the nonlinear behaviour of the structural and the non-structural members, the following assumptions were made:

- All slabs (solid and ribbed) were assumed to behave as rigid diaphragms.
- The stairs were introduced in the model as inclined pinned rods.



- The rigid zones in the joint region of beams/columns were modelled using rigid arms.
- The cross-sections of beams were modelled as flanged cross-sections, with effective flange width provided by the valid Greek codes at the time of construction.
- The perimetrical r/c walls of the basement were modelled using shell finite elements.
- For all structural members, reduced cross-sectional properties were adopted based on the application of paragraph A.3.2.4(5) of EN1998-3 [3].
- For the concrete, the stress-strain model was adopted according to equation (3.14) of EN1992-1-1 [4], taking into account the values of the strength and deformation parameters applicable to concrete grade B225. In addition, in order to take into account the behaviour of the concrete in a cyclic loading, the modified Takeda Model was employed [5].
- For the reinforcing steel, a stress-strain hardening model was adopted (Fig. 3.7a. of EN1992-1-1 [4]) taking into consideration the values of the strength and deformation parameters applicable to steel grade StIIIa.
- The modelling of the non-linear behaviour of the structural members and their damage is based on the commonly used the lumped plasticity model. Therefore, the ends of the beams and columns were assumed as potential plastic hinge positions and thus critical locations for damage occurrence. The values of the total chord rotation capacity  $\theta_{um}$  and of the chord rotation at yielding  $\theta_y$  for the critical zones of r/c members were calculated employing the equations of the Appendix A of EN1998-3 [3].
- For the modelling of masonry infill walls, two approaches were made. In the first approach, masonry fills were considered only as permanent vertical loads. In the second approach, they were taken into account both as vertical loads as well as members with capacity of horizontal resistance to seismic loads. In this case, diagonal non-linear springs (e.g. [6]) were employed for their modelling assuming a bilinear stress-strain model, as described in Appendix C of EN1998-3 [3]. Finally, it must be noted that masonry infill walls were modelled as elements bearing seismic loads only in those bays where there were no wall openings of a considerable area (Fig. 2a).
- The calculation of loads and masses was performed by assuming dead loads G and 30% of live loads Q. For all the slabs, the live load was assumed to be 2kN/m<sup>2</sup>, while cantilevers were assumed as having 5kN/m<sup>2</sup>.



Fig. 2 – The FE model and the first three natural periods of the building (a) with and (b) without the influence of masonry infill walls

With regard to the behaviour of the foundation and the soil, the following assumptions were made:

• The structural elements of the foundation (foundation beams (Fig. 1d)) were modelled using linear beam elements, assuming the absence of plastic deformations.



- The soil foundation was modelled on the basis of a discrete model of the dynamic interaction of massive surface foundation supported by a homogeneous, isotropic, linear elastic half-space [7]. Consequently, soil-structure interaction (SSI) phenomena were taken into account. The soil was considered to range between categories A and B, on the basis of the soil categorisation of EN1998-1 [2]. However, with respect to the parametric analyses performed, three different values for the shear wave velocities in the ground (v<sub>s</sub>=250, 450 and 650m/sec) were considered, as will be presented in paragraph 4.1.
- The soil surrounding a section of the basement perimeter (Fig. 1d) was modelled using horizontal elastic springs, which were given spring constants compatible with the data corresponding to the model also used for the modelling of the vertical support.

The models employed with and without considering of the masonry infill walls are presented in Fig. 2. In the same figure the first three natural periods of the building are also presented.

## 3. Seismological data

The seismic sequence in the island of Cephalonia, in the beginning of 2014, consisted of a large number of seismic excitations of various magnitudes. However, the main characteristic of this sequence was the two mainshocks on 26/01/2014 ( $M_w=6.1$ ) and on 03/02/2014 ( $M_w=6.0$ ). As a result, in a timespan of one week, the buildings and the infrastructures in the island suffered two strong seismic events which in many cases caused heavy damages [1]. The sequence was monitored and recorded by both the permanent and temporary accelerograph network of EPPO-TSAK, as well as the one of the NOA [1]. The accelerograms which were used in the present paper (Fig. 3b) were recorded by a station of the NOA (first main event) and by a station of EPPO-ITSAK (second event). Both stations were installed in the Lixouri Town Hall located in a 700m distance from the analysed building (Fig. 3a) [1].



Fig. 3 – The location of the analysed building and the recording instruments [source: Google Earth] (a) and the recorded accelerograms (b)



Fig. 4 illustrates the elastic response spectra (structural damping ratio 5%) and the fundamental period  $T_1$  of the building, as well as the values of characteristic seismic parameters for the first and the second mainshocks.





	FIRST EVENT		SECOND EVENT	
	N-S component	E-W component	N-S component	E-W component
PGA [m/sec <sup>2</sup> ]	4.650	5.690	5.925	6.589
PGV [m/sec]	0.318	0.616	0.810	1.167
PGD [m]	3.610	2.260	0.360	0.830
PGV/PGA [sec]	0.068	0.108	0.137	0.177
Arias Intensity [m/sec]	2.680	3.106	1.975	4.138
Housner Intensity [m]	1.220	1.890	2.822	4.817
<b>Characteristic Intensity</b>	3.404	3.800	2.442	4.253
Perdominant Period [sec]	0.300	0.660	0.380	1.360
Mean Period [sec]	0.351	0.568	0.939	1.235
Uniform Duration(0.16g) [sec]	1.860	2.120	1.060	2.095
Bracketed Duration(0.16g) [sec]	6.310	6.960	4.515	4.035
Significant Duration [sec]	9.250	10.730	5.735	4.200

(b)

Fig. 4 – The elastic response spectra (a) and the values of seismic parameters of the two strong events (b)

In Fig. 5, the orientation of the two horizontal components of the mainshocks' records (in the N-S and the E-W direction) was illustrated with respect to to the constructional axes of the analysed building.



Fig. 5 – The orientation of the horizontal components of the two mainshock records and of the constructional axes of the analysed building



# 4. Analyses procedure and results of the investigation

### 4.1. Analyses procedure

In order to achieve the goals of the present paper, non-linear time response analyses (NTRA) were utilized. The accelerograms which are presented in fig. 3 were used as seismic inputs. The two horizontal components of seismic excitations in the N-S and E-W direction were considered to be parallel to the constructional axes of the building X and Y respectively (Fig. 5). As regards the analyses for the seismic sequence, new accelerograms were created. These accelerograms consist of the accelerograms of the first and the second mainshocks separated by a time gap equal to 100sec, according to suggestions which are given in the literature (e.g. [8]).

Due to the fact that the building's structural parameter values (the materials' properties, the dimensions of the structural members and their corresponding reinforcement amounts, the vertical live and dead loads etc.) were accurately known ("full knowledge level" according to EN1998-3), the parameters investigated in order to establish the observed level of damage were the influence of the masonry infill walls on the seismic response (using the FE model of the Fig. 2a), and the influence of flexibility and damping parameters of the soil foundation. From measurements which were available for the greater area of Lixouri, the shear wave velocity  $v_s$  in the ground was estimated as greater than 300m/sec. However, due to the fact that more accurate and localized measurements were not available in the region of the analysed building, it was decided to perform analyses using three different values for the shear wave velocity in the ground:  $v_s=250$ , 450, 650m/sec.

With regards to the assessment of the damage state of the analysed building, the decision adopted was to use the chord rotation  $\theta$  ( $\theta$ =the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span i.e. the point of contraflexure), as recommended by EN1998-3 [3]. In addition, the Maximum Interstorey Drift Ratio (MIDR) was used as a damage state classification criterion. The MIDR is widely employed as parameter for seismic damage level assessment (e.g. [9]).

### 4.2. Analyses results

Figure 7 illustrates the distribution of critical zones (ends) of beams and columns respectively according to the values of chord rotations as they were calculated from the NTRA. In order to model the non-linear behaviour of the r/c members, as well as to estimate the characteristic values of chord rotation  $\theta$  (on the basis of the equations of appendix A of EN1998-3), the assumptions which are presented in Fig. 6 were made.



Fig. 6 - Basic assumptions for the definition of the damage state of r/c members

From Fig. 7, it can be observed that the estimation of the damage level of r/c members is accomplished through the comparison of the calculated values of chord rotation at the critical zones with characteristic values of chord rotation which correspond to the predefined damage levels. When  $\theta < \theta_y$ , no damage occurs. When  $\theta_y < \theta < 0.1\theta_{um}^{pl}$ , almost invisible damage occurs, which does not need any consideration (slight damage). When  $0.1\theta_{um}^{pl} < \theta < 0.25\theta_{um}^{pl}$ , damage is repairable and is not characterized as serious (moderate damage). When  $0.25\theta_{um}^{pl} < \theta < 0.55\theta_{um}^{pl}$ , damage is considered serious and needs special and costly techniques in order to be repaired (heavy damage). Finally, when  $\theta > 0.55\theta_{um}^{pl}$ , damage is characterized as not repairable.



Fig. 7 – Distribution of the critical zones of beams (a) and columns (b) in the predefined damage levels

On the basis of the previously described criteria for the damage classification, it can been seen from Fig. 7a that for all examined cases, the influence of the second mainshock in the increase of the damage in beams is very significant. For example, it can be mentioned that from the total of 158 locations of potential formation of plastic hinges in the critical zones of beams, in the case of foundation soil with  $v_s=250$ m/sec, the number of critical zones in which moderate or heavy damage occurs  $(0.1\theta_{um}^{pl} < \theta < 0.55\theta_{um}^{pl})$  increases from 11(=11 critical zones with moderate damage and none with heavy) to 30 (=24 critical zones with moderate damage and 6 with heavy). This conclusion is drawn from the analysis in which the model with the influence of the masonry infills is used (Fig. 2a). Moreover, during the second mainshock, critical zones with heavy damage occurred as well  $(0.25\theta_{um}^{pl} < \theta < 0.55\theta_{um}^{pl})$ . Corresponding results were also obtained from the analyses in which the two other soil categories were considered.

The most significant parameter for the interpretation of the low damage level which was observed in the analysed building after the second mainshock is the participation of the masonry infill walls in the seismic response of the building. From the analyses performed can be concluded that if the participation of masonry infill walls to the building's seismic response is not taken into account, the overall predicted damage state is totally different and significantly severe than the damage level which results if this participation is considered (Fig. 7a). Indicative of this conclusion are the results of the case in which the foundation soil with  $v_s$ =450m/sec



was considered. In this case, when the masonry infill walls are imported to the FE model, the number of the critical zones where the damage is heavy ( $\theta$ >0.55 $\theta_{um}^{pl}$ ) after the second mainshock, is 6 (Fig. 7a). The corresponding number when the masonry infill walls are ignored is 67 (=45+22). In other words, by ignoring the masonry infill walls in the analyses, we obtain an unrealistic damage state which was not observed during the insitu inspection after the second mainshock. Corresponding results are also produced from the analyses in which the two other soil categories are considered.

With regards to the influence of the soil foundation category on the damage state, the conclusion is that when a soil with  $v_s$ =650m/sec was chosen, the analytically calculated damage level approaches more the corresponding observed level. However, it must be noted that the differences in the overall damage state are insignificant when other soil category is selected (especially when the selected foundation soil is characterized by shear wave velocity  $v_s$ =450m/sec).

The conclusions which turn out from Fig. 7a (which refers to the damage levels of beams) are generally identical with the corresponding conclusions for the columns' damage which are presented in Fig. 7b (in buildings' columns, there are 122 critical zones, i.e. potential locations of plastic hinges formation, in total). However, in the case of columns, the damage level is significantly lower than the aforementioned damage levels of beams. For this reason, these conclusions are not clearly observable from Fig. 7b. In any case, the conclusion according to which the influence of the masonry infill walls possesses the basic role in the observed low damage state still holds. It must be noted here, that the significantly low damage level of columns is also due to the fact that the dimensions of their cross-section, as well as their reinforcement (see paragraph 2.1), correspond to columns in close distances, especially in the perimeter of the building, which leads to an optimal distribution of vertical and horizontal seismic loads.





Figures 8 and 9 illustrate the Maximum Interstorey Drift Ratios (MIDRs) of the ground floor and the 1<sup>st</sup> floor in the directions of the constructional axes X and Y for all analyzed cases. From these figures, it can be deduced that the maximum relative displacements of storeys are increased in all cases after the second mainshock. This conclusion is obviously due to the fact that the building suffers damage (even if extremely minor) mainly occurred in masonry infill walls after the first mainshock, as well as due to the fact that the second mainshock has a magnitude of the same order as the first. Therefore, the building responded to the second mainshock having lost a small amount of its structural members and masonry infill walls stiffness. In particular, with regards to the masonry infill walls, the following remarks can be made:

(a) When the masonry infill walls are taken into consideration in the FE model as members which receive horizontal seismic loads, the MIDRs take values between 0.50% and 0.75%. These values can explain the low damage level after the first mainshock (e.g. [9]).

(b) The favorable influence of the masonry infill walls is more significant during the seismic sequence. More specifically, the combined study of Figs. 8 and 9 leads to the conclusion that when the masonry infill walls are



ignored in the FE model, all MIDRs take values greater than 1.0% (in most cases greater than 1.5%, while in some cases even greater than 2%). These values can lead to the misleading conclusion that the building was suffered heavy damage, something that was not confirmed from the in-situ inspection after the second mainshock.



Fig. 9 – MIDRs of (a) the ground floor and (b) the 1<sup>st</sup> floor along the constructional axis Y of the building

The conclusions about the influence of the foundation soil category on the damage state when the MIDR was used as the damage level criterion are similar to those extracted when the damage level was assessed using the chord rotation in the critical zones of structural members. More specifically, the soil foundation category does not affect significantly the damage state of the building after the first or the second mainshock. To illustrate more precisely, it can be noted that the comparison of the results obtained from the analyses in which soils with  $v_s=450m/sec$  and  $v_s=650m/sec$  were respectively used led to the conclusion that these results are almost similar, especially in the case in which the masonry infill walls are considered.

Apart from the damage assessment using the chord rotation in the critical zones of the structural members and the MIDRs, the investigation of the sufficiency of the shear resistance of columns was considered as necessary. In spite of the fact that no shear failure in columns was observed, the verification of the sufficiency of columns against this type of failure was utilized as an additional check of the effectiveness of the FE models used. For this verification, the results from the more effective FE model of the building (i.e. the FE model in which the masonry infill walls were included and a soil with  $v_s$ =650m/sec was taken into consideration) were used. The eq. (A.12) in Appendix A of EN1998-3 was applied for the estimation of the shear resistance.

	First event			Sequence				
	Column	maxV(t)	V <sub>R</sub>	maxCR	Column	maxV(t)	V <sub>R</sub>	maxCR
Basement	C18	99.3	218.8	0.45	C12	120.7	245.0	0.49
<b>Ground floor</b>	C19	144.3	200.7	0.72	C16	154.0	163.6	0.94
1 <sup>st</sup> floor	C20	116.3	117.0	0.99	C20	116.3	117.0	0.99

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The above table illustrates the maximum shear capacity ratios ( $CR=maxV(t)/V_R$ ) of columns of the basement, the ground floor and the 1<sup>st</sup> floor. These ratios correspond to the maximum shear force obtained during the first mainshock, as well as during the seismic sequence. From this table, it can be seen that no shear failure was occurred. These results verify the in-situ damage inspection conducted after the second mainshock.

4.3. Comparison of analyses results with the observed damage state

In the current paragraph, an estimation will be made for the level in which the analytically calculated damage state approximates the actual damage state observed during the in-situ inspection conducted after the second mainshock.



Figure 10 illustrates the results of the in-situ damage inspection after the second mainshock. Due to the fact that the time span between the two mainshocks was only eight days, damage inspection was made only after the second mainshock. Therefore, the comparisons between the analytically calculated and the actual damage state concern the sequence of the two mainshocks. It must be noted that no damage in structural r/c members was detected, except for some surface cracks in the concrete. As presented Fig. 10, various types of damages were detected in masonry infill walls. The importance of these damages ranges from simple coating cracking to bidiagonal cracking and/or masonry detachments. Fig. 10 also yields the conclusion that the masonry damages were detected are more in number and more serious in the ground floor. This figure also shows that the more serious damages occur in the masonries along the structural axis Y.



Fig. 10 –The results of the in-situ damage inspection of the examined building after the second mainshock for the 1<sup>st</sup> floor (a) and for the ground floor (b).

The conclusions reached from Fig. 10 can be generally evidenced by the results of the analyses which are made in the framework of the present paper. As regards the conclusion that more masonry damages appear in the ground floor than in the 1<sup>st</sup> floor, it must be reminded that Figs. 8 and 9 show that, in general, the MIDRs which result from the seismic sequence (from the FE model in which the influence of the masonry infill walls is taken into consideration) are greater in the ground floor. Another conclusion which is drawn from the study of Figs. 8, 9 and 10 is that the MIDR values which better validate the observed damage state result from the FE model in which a soil with  $v_s=650$ m/sec is considered. This FE model leads to a maximum MIDR value of 1.08% in the ground floor, whereas the FE model in which a soil with  $v_s=250$ m/sec is considered leads to a maximum MIDR value of 1.23%. Obviously, the observed masonry damage better match the MIDR value of 1.08% than the value of 1.23% at the ground floor.

### 5. Conclusions

The present paper attempts to investigate the parameters which caused the observed damage state in an r/c building in the town of Lixouri in the island of Cephalonia (Greece) during the seismic sequence at the beginning of 2014. The distinctive characteristic of this sequence is that it included two strong events in a short time span of eight days. The examined building is a low-rise r/c building, which was constructed in 1985 under the valid Greek codes applying at that time. The in-situ damage inspection after the second mainshock detected various types of damage in the masonry infill walls, both serious and minor. No damage was detected in the r/c structural members, except for some surface cracks in the concrete.

In order to achieve the goals of this investigation, non-linear time history analyses were utilized. Two FE models were created for the examined building: in one of them, the influence of the masonry infill walls on the seismic response of the building was considered, whilst in the other the latter influence was ignored. Furthermore, the soil-foundation-structure interaction was investigated by using a foundation model which considers SSI phenomena. Three soil categories were considered based on the speed of seismic shear waves  $v_s$  ( $v_s$ =250, 450, 650m/sec). The formation of the FE models, as well as the analyses procedure, followed the recommendations of EN1998-3.



The assessment of the analytically calculated damage was performed by using the chord rotation capacity of the critical zones of the r/c members (as EN1998-3 suggests), as well as by using the Maximum Interstorey Drift Ratio (MIDR), which is a well-known criterion for the damage assessment. Furthermore, the sufficiency of columns against shear failure was checked. The basic conclusions which were extracted from the investigation described above are the following:

- (a) The influence of the second mainshock on the overall damage state was very significant. The plastic deformations in the critical zones of the r/c members, as well as the values of the MIDRs, were increased significantly after the second mainshock as compared to their values after the first mainshock.
- (b) The existence of strong masonry infill walls consists the main factor which led to the low overall damage level which was observed, although the building suffered two strong seismic events (with PGA equal to 0.54g and 0.68g respectively) in the short time span of eight days.
- (c) The soil conditions affected the observed damage level but not to the same extent as the masonry infill walls. However, the results of the analyses confirmed the available measurements according to which the soil in the greater area of the building belongs to category B ( $v_s$ =360-800m/sec) on the basis of the classification of EN1998-1.
- (d) The FE model of the building which includes the masonry infill walls and takes into consideration a soil with  $v_s=650$ m/sec very closely approaches the actual damage state which was observed after the second mainshock. This approach is utterly satisfactory in the case of columns, but not as satisfactory in the case of beams. As regards the beams, the optimum FE model predicts higher damage levels than the actual ones, but the differences are not significant. These deviations might be attributed to many factors, such as the utilized model of the masonry infill walls which is based on non-linear bi-diagonal rods (which can't simulate the masonry behavior accurately), the fact that the reinforcement details of beams weren't available, and finally the fact that the reinforcement of slabs which is located into the effective flange width of beams was ignored in the calculation of their yielding moments, as well as their ultimate (strength) moments.

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