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Nonlinear seismic assessment of eight-storey reinforced concrete building according to Eurocode EN 1998-3

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ABSTRACT

In the present article, the nonlinear seismic assessment of an irregular 8-storey reinforced concrete building according to Eurocode EN 1998-3 (2005) and the recent Greek Code for Retrofitting of Reinforced Concrete Buildings is presented. The assessment was prompted from safety concerns generated by a planned removal of all the infill walls in the first story and their replacement by light-weight partitions. What makes this study interesting is: (a) the fact that the building had been repaired and strengthened after an M=6.0 damaging earthquake in 1986, and (b) the availability of the strong motion records from that earthquake from an instrument at the building basement. The seismic capacity assessment of the building is based on nonlinear dynamic and static pushover analyses, taking into account the increased seismic safety levels imposed by the current Codes. Modeling issues pertaining to the limit states of reinforced concrete members, e.g. member end rotations at yield, full plastification and failure, effective member stiffness for nonlinear analyses, are discussed and the selections made for this work are presented. Infill walls are modeled using simple strut members working only in compression. The required properties were taken from available drawings, but were also verified by in situ measurements. The study indicated that removal of the first story infill walls had no consequences on the seismic safety of the building but also showed that if the building is hit by a current design level earthquake, some damage might be expected in the upper floors at one side of the building.

1. INTRODUCTION

The investigation reported herein was carried out for the administration building of the Messinia prefecture, an 8-story structure in the city of Kalamata in southern Greece (Fig. 1). This building has an elongated rectangular shape, with dimensions of its framing plan L=38.4m and B=13,35m. In the middle of the building there is an expansion joint perpendicular to the long direction, acting also as a seismic separation gap. Since the building is symmetric with respect to the axis of this joint, only the left

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half of it was considered (Fig. 2) This building was retrofitted after been damaged in an M=6.0 earthquake that struck the city in 1986 (Anagnostopoulos *et al*, 1987).The need for assessment of its seismic capacity arose when plans were announced to remove all the masonry infill walls in the first story and replace them with light partitions. There were certain concerns by various employees for this change, based on the argument that the building would be weakened. This prompted the investigation of the seismic capacity of the building as is and the effects, if any, of removing the first story partition walls. Thus two models of the building were created and investigated, with and without infill walls in the first story, while everything else remains the same.

Fig. 1 The eight-story R/C building in Kalamata

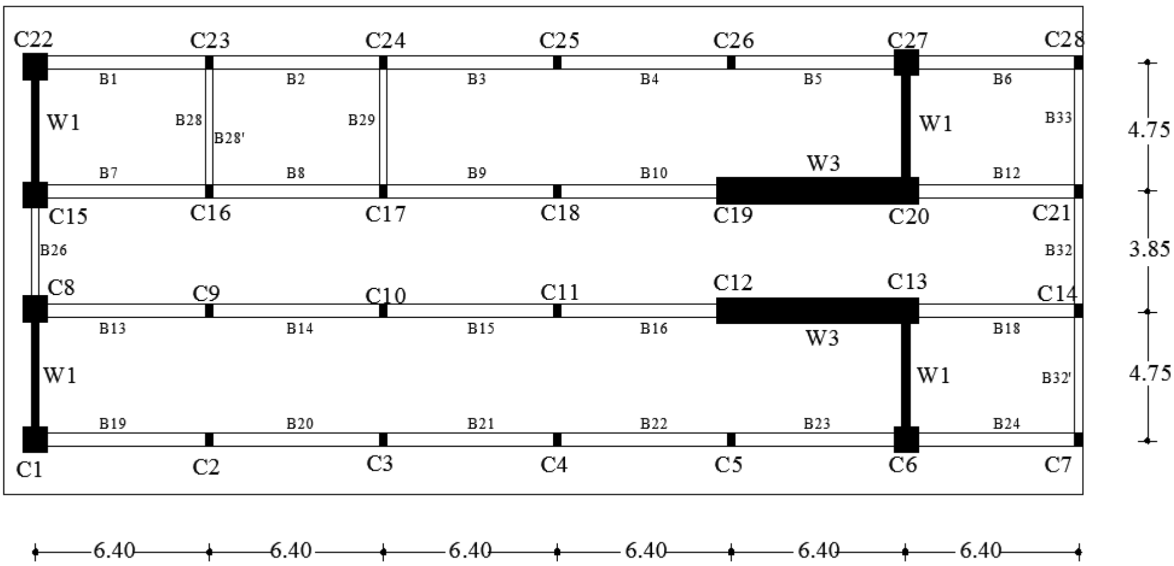


Fig. 2 Layout of the eight-story r/c building (dimensions in meters).

Correct modeling of a structure is always a prerequisite for correct numerical results. Such modeling must include members and elements that may not always be characterized as structural, e.g. brick infill walls, which, however, possess significant initial stiffness and strength. When nonlinear analyses are used for capacity assessment, modeling requirements are substantially more complicated and difficult compared to those for elastic analyses, given that one must calculate (1) Moment-Chord Rotations ($M-\theta$ diagram) at each critical end of each structural member, (2) available cyclic shear strength of each structural member, which determines the expected mode of failure, ductile or brittle, of the member, (3) effective member flexural stiffness, (4) moment-axial force interaction diagrams for beam-column elements, and (5) initial stiffness and strength of the masonry infill walls. The first four of these items are fully covered by Eurocode EN 1998-3 (2005) and the recent Greek Code for Retrofitting of Reinforced Concrete Buildings (KANEPE, 2012), while for the infill brick walls information from the literature has been used (Makarios, 2013).

(a) (b) (c) (d) (e) (g)

Fig. 3 (a,b) Magnetic tests for detection of steel bar location (c,d) Concrete strength measurement (e) Floor slab coring (g) Detection of concrete carbonation

2. DATA AND MODEL OF THE EIGHT-STOREY R/C BUILDING

In order to verify the design assumptions and available drawings, some coming from the original design and some from the retrofitting phase, a series of non-destructive in situ tests was carried out (Fig. 3). The results indicated a characteristic concrete strength of 17.25 MPa, mean strength 20.50 MPa and Young's modulus E = 26.00 GPa. The mean steel strength is 545 MPa and the maximum strength 794 MPa, reached at a strain of 10% plus. The steel strain at failure was found to be 18%.

The elastic and inelastic analyses of the building were carried out with the finite element program SAP 2000. The 3D geometric model of the building is shown in Fig. 4. Floor masses *m* and mass moments of inertia J_m were calculated analytically and are listed in Table 1 for each floor. In the same Table, the floor elevations z from the ground reference level are also given.

The idealization of the masonry infill walls is made with equivalent diagonal truss elements, as shown in Fig. 5 for the building model with the first story infills removed.

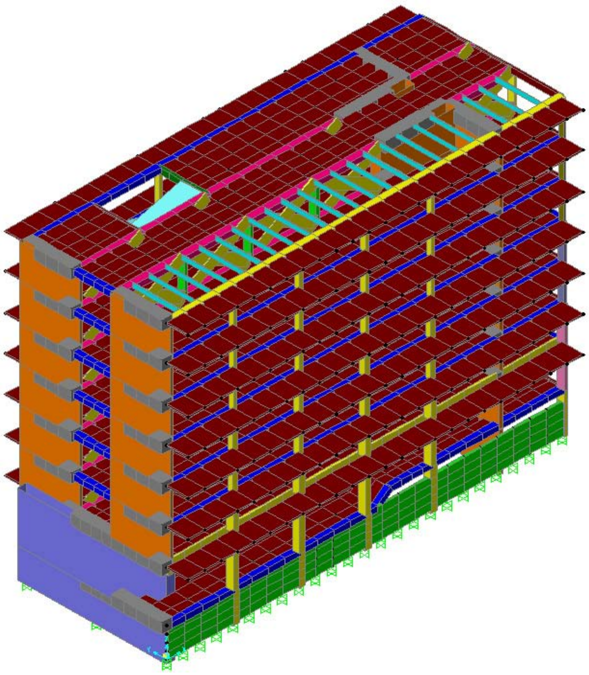


Fig. 4 3D model of the eight-storey r/c building

Table 1 Floor elevation z (m), masses *m* (tons) and floor torsional mass-moments of inertia J_m ($tons \cdot m^2$)

Fl.	1	2	3	4	5	6	7	8
z	2.84	7.65	11.00	14.42	17.81	21.18	24.67	27.98
m	774.48	1015.74	937.45	950.59	919.2	905.61	1112.63	794.95
J_m	103806	146747	135437	133002.	132800.	130837.	160745.	100262

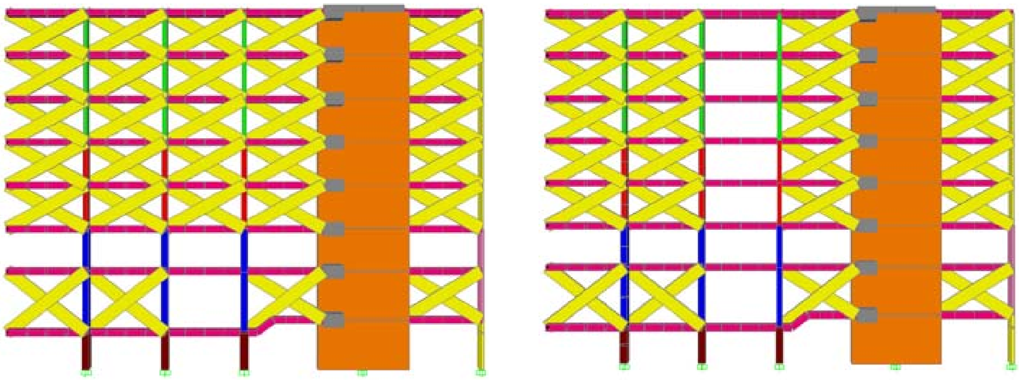


Fig. 5 Infill wall modeling with first story infills removed:
The two internal frames in the longitudinal direction

Table 2 The first three periods of the eight-storey building

	T_1	T_2	T_3
Model 1: Building as is	0.89014 s	0.63154 s	0.58167 s
Model 2: First story infills removed	0.89025 s	0.634089 s	0.581984 s

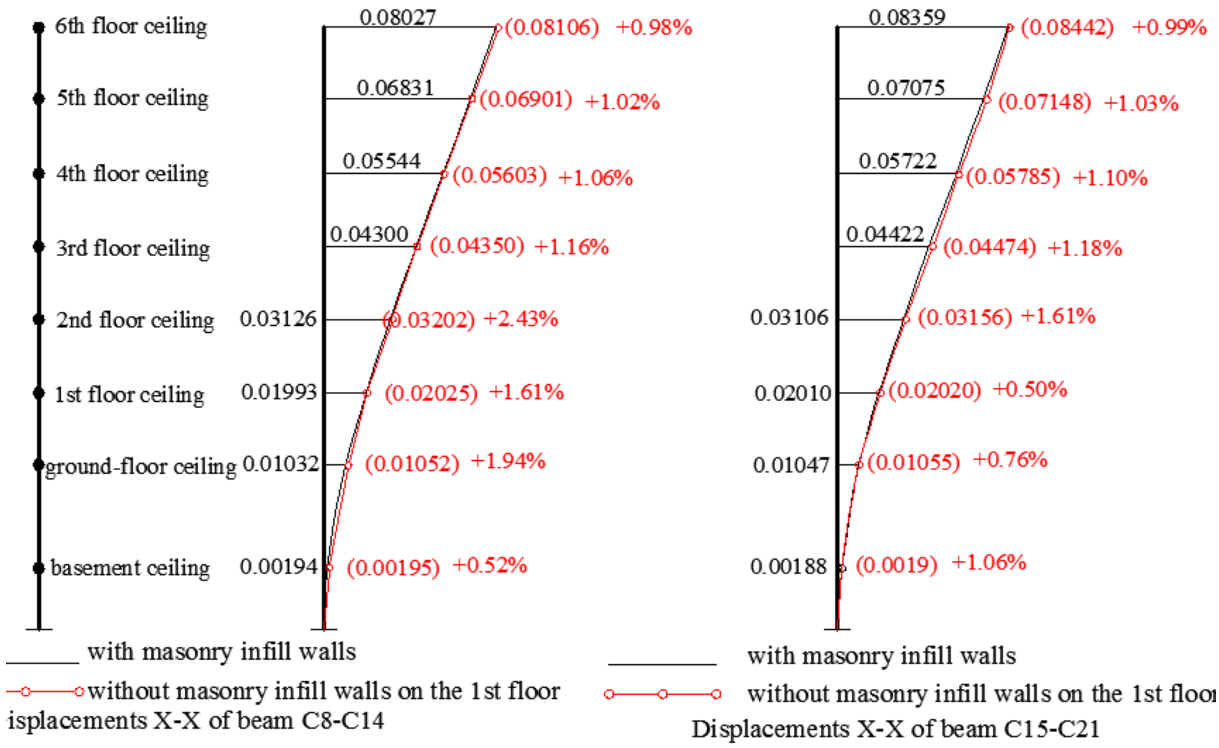


Fig. 6 Maximum floor displacements by response spectrum analysis
Comparison of the two models (floor elevations in Table 1)

3. RESPONSE SPECTRUM ANALYSIS

The response spectrum analysis was carried out for ground acceleration A=0.24g, (g = acceleration of gravity) applicable to the seismic zone for Kalamata, as per the current code for new buildings, and the elastic design spectrum of Eurocode 8 for soil category *D*. Table 2 shows the lowest 3 periods of the two models, which are, as expected, almost identical. This indicates a practically zero influence of the first story infills on the building's stiffness. The same conclusion is reached by looking at the maximum floor displacement profiles of the two models, computed by response spectrum analysis for the EC-8 design spectrum (Fig. 6).

Moreover, comparing the elastic bending moment of the columns for the two aforementioned models, all the differences are below 6%. Thus, one can conclude that the elastic building response is practically unaffected by the removal of the first story infills. The same analysis indicates that bending moments in columns C5 (on ground floor), C7 C11 C14 C18 C24 (on the sixth floor), and C21 in all levels, as well as in several beams exceed the corresponding bending strengths, which points to the need for further investigation using inelastic analyses.

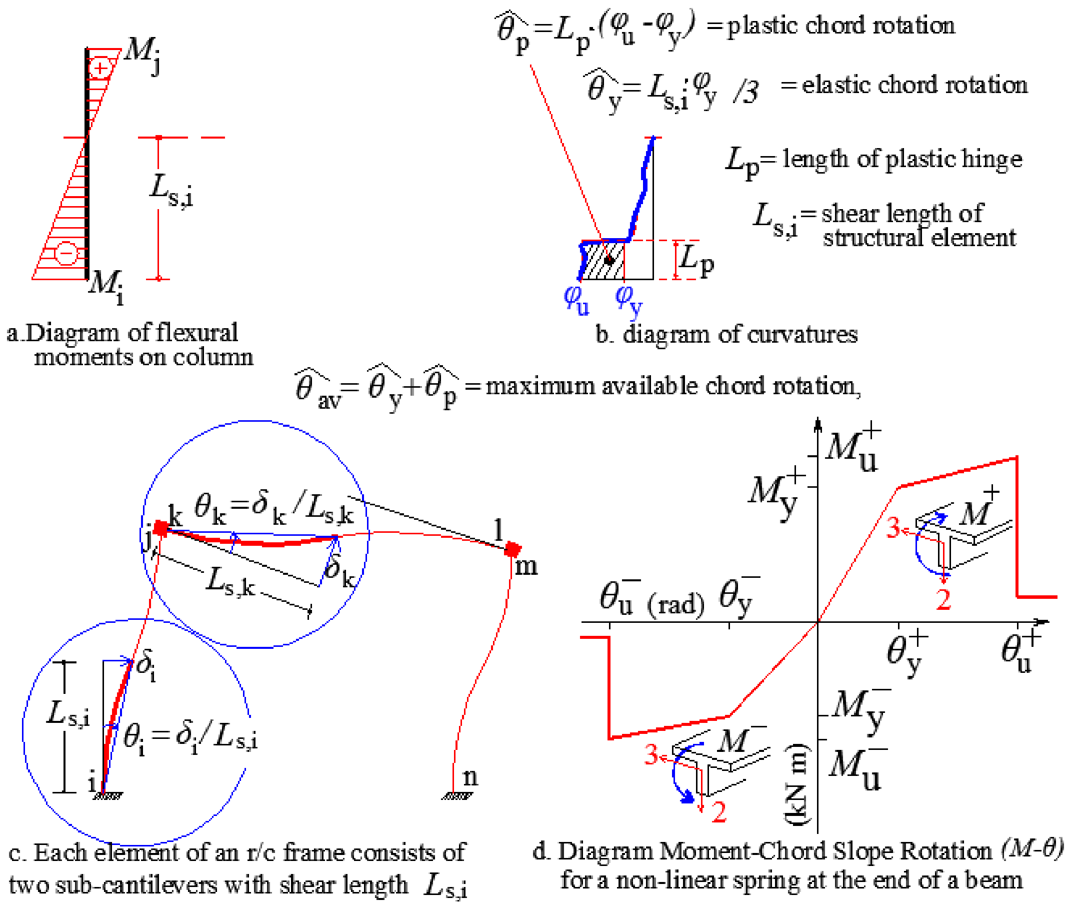


Fig. 7 Inelastic beam modeling for the SAP2000 program

4. NONLINEAR SIMULATION OF STRUCTURAL R/C MEMBERS

Modelling of the concrete beams and columns was carried out as required by the non-linear computer program SAP2000 and detailed in another paper (SAP2000, 2011, Makarios, 2012). The so called one-component, plastic hinge model is used, where each prismatic flexural member is idealized with three sub-elements: the elastic prismatic beam or beam column and two nonlinear springs of zero length attached at the two ends of the elastic element. In order to find the characteristics of a plastic hinge, it is assumed that each member (beam/column) deforms in antisymmetric bending. The required "shear length, $L_{s,i}$, is determined from the point of contra-flexure (Fig. 7), while the Moment-Chord Slope Rotation ($M-\theta$) diagrams of the non-linear springs are determined on the basis of the element properties (concrete section, and reinforcement properties) using a special purpose program based on the fiber model (XTRACT, 2007)

In order to calculate the $M-\theta$ diagram of a plastic hinge, two methods can be used: (a) one as proposed by Eurocode EN-1998.03 (Annex A: sections from A.3.2.2 until A.3.2.4 and derived from a large amount of experimental data (Panagiotakos & Fardis 2001). (b) using a moment-curvature (M - φ) diagram for numerical computations by means of a fiber model of the member, as implemented in various computer codes (XTRACT, 2007; Section-Designer /SAP2000v15, 2011) This requires a suitable length L_p for each plastic hinge (Fig.7b). In these calculations the limiting yield, plastic and ultimate rotations θ are computed as follows

$$\left.\begin{array}{c} \theta_y=\varphi_y \cdot L_s/3 \\ \theta_p=(\varphi_u-\varphi_y) \cdot L_p \\ \theta_u=\theta_y+\theta_p \\ L_s=M/V \end{array}\right\}$$

where φ_y , φ_u are the yield and ultimate curvature of the end section, respectively, L_s the shear length and L_p the plastification length with assigned values based on experimental results (e.g. for a beam taken equal to its depth). Additional information on the plastification length for different concrete structural members can be found in the literature (e.g. Panagiotakos & Fardis ,2001, Salonikios; 2003, Paulay & Priestley 1992). M is the elastic flexural moment and V the corresponding shear force at the member end due to earthquake. For tall-walls, the "shear length" L_s can be calculated as the distance , in elevation, of the zero-moment point to the basefor a set of lateral, earthquake static floor forces. For the present paper M-θ diagrams were calculated using the program XTRACT. Note that our calculations did not consider sources of inelastic behavior such as slippage of reinforcement or opened cracks with yielding steel bars.

Masonry infill walls were modelled for our analysis using diagonal struts working only in compression (Fig. 8) in accordance with the new Greek code for retrofitting existing concrete structures (KANEPE 2012). More details can be found in Makarios, 2013.

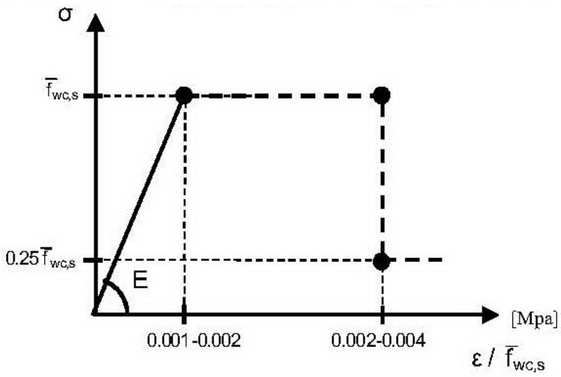


Fig. 8 Stress-strain diagram for diagonal compressed inelastic bar (Masonry infill walls)

5. NON LINEAR DYNAMIC RESPONSE HISTORY ANALYSES (NLDA)

For the nonlinear analyses, the flexural stiffness of the various beams were based on secant stiffness at yield, which according to modern codes for retrofitting existing structures, e.g. KANEPE 2012, EC8-3, is only a small fraction of the flexural stiffness EI for elastic analyses. For an idea of the resulting differences in the overall stiffness of the building, the periods corresponding to secant member stiffness were computed and listed in Table 3. Comparison with the corresponding periods in Table 2 indicates that the inelastic models are 70% and 52% softer than the model used for the response spectrum analysis (~ elastic) along the y and x axes respectively.

The nonlinear response history analyses (NLDA) were performed using three pairs of semi-artificial motions, compatible with the Design Spectrum of Eurocode EN 1998, generated using the method by Karabalis et al, 1994. Additionally, the recorded motion at the base of the building from the 1986 earthquake was used (Anagnostopoulos *et al*, 1987) for comparison and verification purposes. Fig.9 shows the code design spectrum explained previously, the response spectra of the semi-artificial motions and the response spectra of the two horizontal components of the 1986 records at the building basement.

Table 3 Periods and effective modal masses for the two models of the building

Mode shape	**With masonry infill walls**			**Without masonry infill walls of the 1st floor**		
	T (sec)	***M****$_x$ (%)	***M****$_y$ (%)	***T*** (sec)	***M****$_x$ (%)	***M****$_y$ (%)
1	1.05159	0.20	54.86	1.05168	0.21	54.83
2	0.87551	66.62	0.35	0.88446	66.77	0.36
3	0.63162	0.36	6.72	0.63172	0.33	6.73
4	0.23759	0.00	20.14	0.23759	0.00	20.13
5	0.19085	15.83	0.02	0.19128	15.86	0.02
6	0.18405	1.59	0.03	0.18410	1.40	0.03

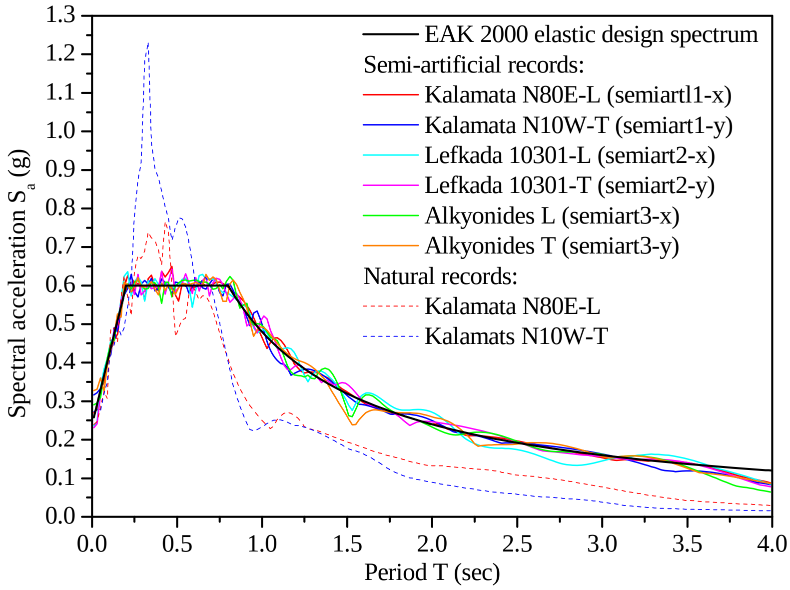


Fig. 9 Response spectra of the semiartificial motions and of the Kalamata records and elastic acceleration spectra of used accelerograms

The response history of the building top (center of mass) along the longitudinal (x) direction for both models under the two component real Kalamata record is shown in Fig. 10. The influence of the first story infills is negligible. The same conclusion, i.e. negligible influence of the first story infill walls, can be observed in the maximum displacement profiles for the three artificial motion pairs (Fig. 11) and in the interstory drifts (Fig. 12) shown as percentages of the story heights.

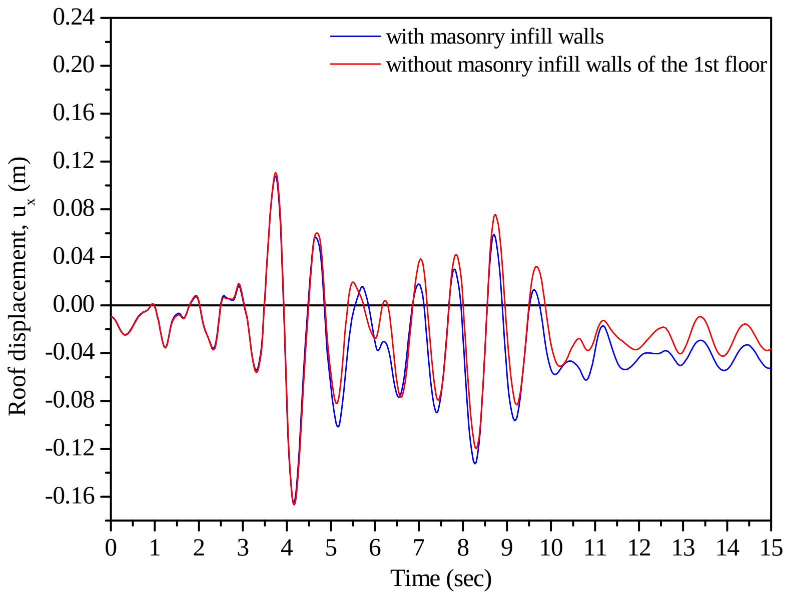
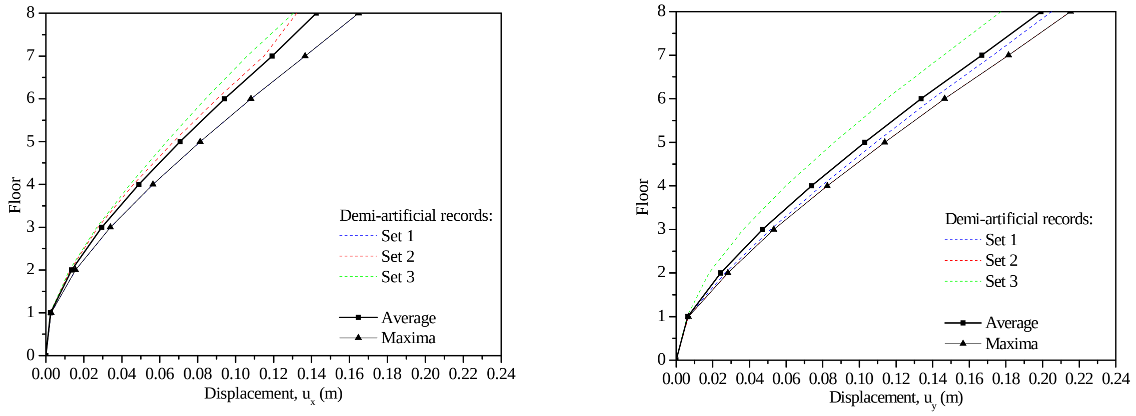
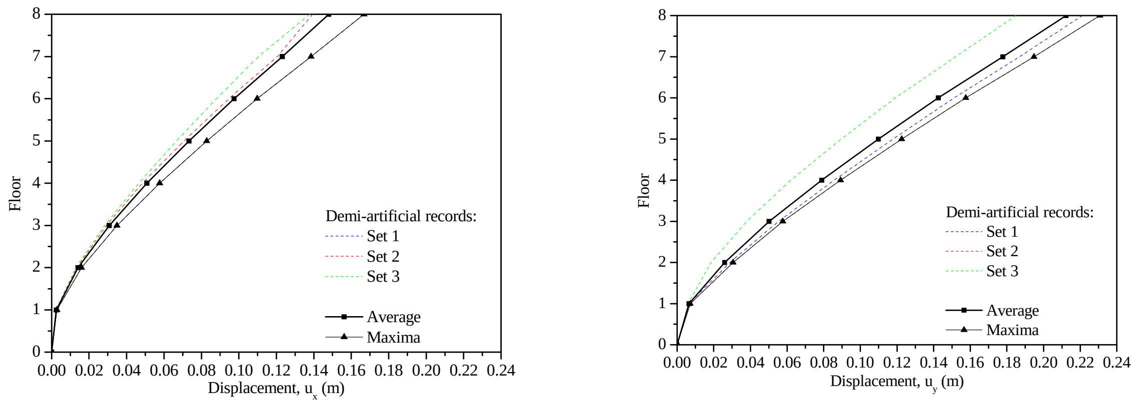


Fig. 10 Response history of center of mass at the building top under the action of the real, two component, Kalamata record

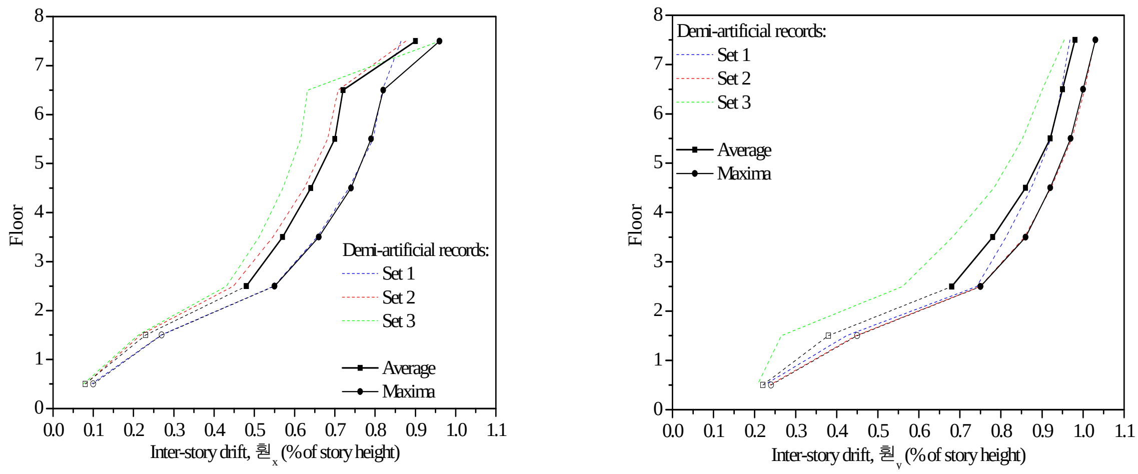


With masonry infill walls of the first floor

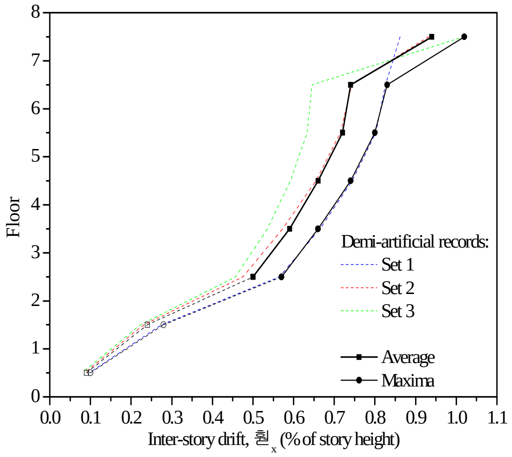


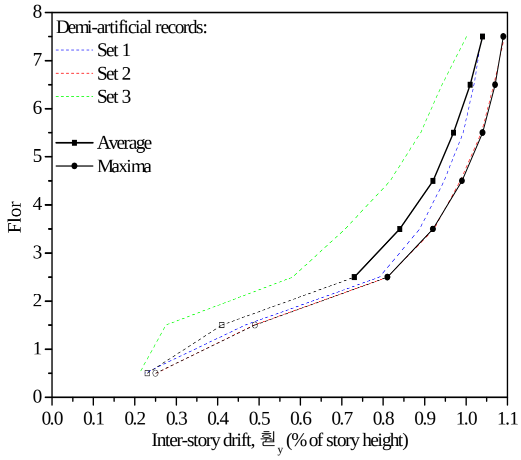
Without masonry infill walls of the first floor

Fig. 11 Maximum floor displacements of the two models



With masonry infill walls of the first floor



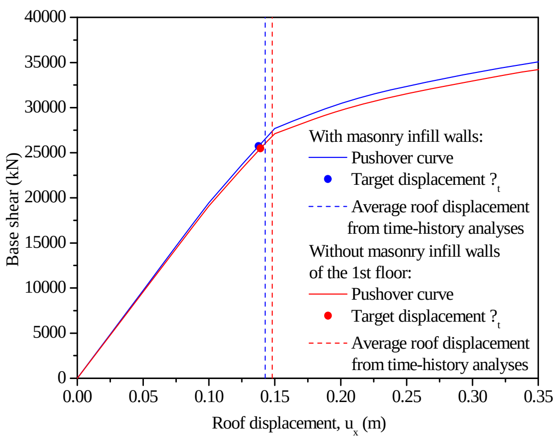


Without masonry infill walls of the first floor

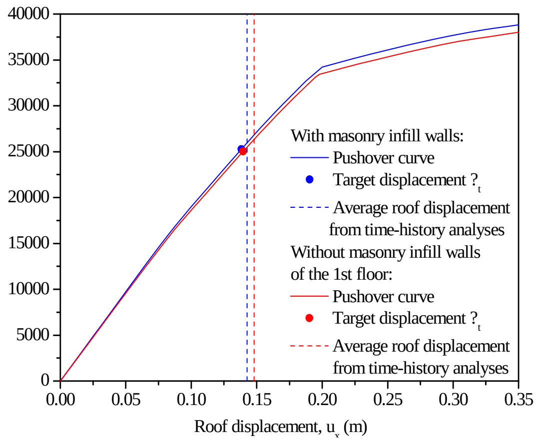
Fig. 12 Interstory drifts (% of story height) for the two models

6. STATIC PUSHOVER ANALYSIS

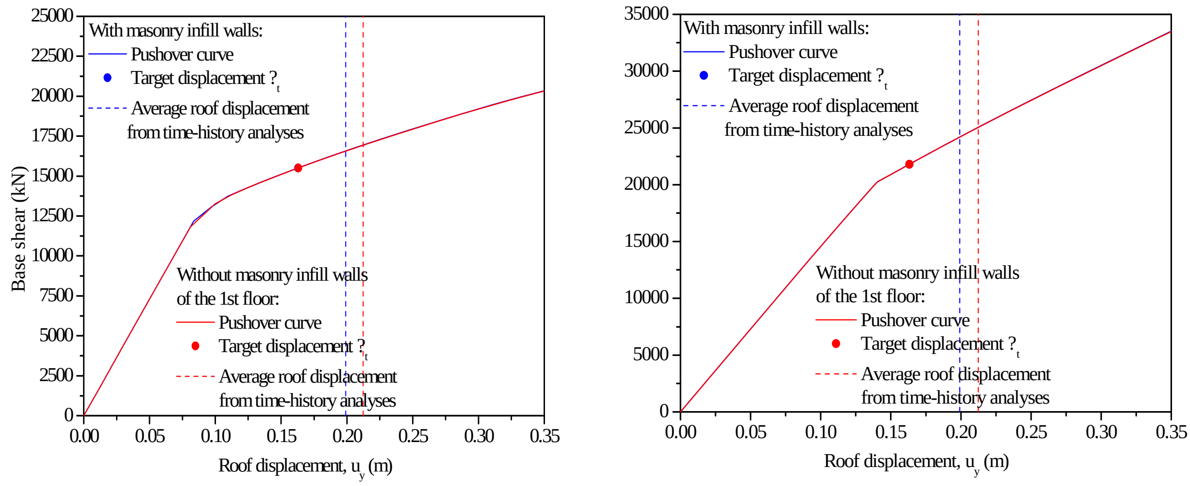
In addition to the non linear dynamic analyses, static nonlinear (pushover) analyses have also been performed for investigation reasons. The required target displacements were estimated as suggested by the code and for comparison also as the mean peak displacements from the NLDA solutions. The agreement, as it can be seen in Fig. 13 is quite satisfactory. From these pushover curves, as well as from the graphs of corresponding interstory drifts, Fig. 14, it becomes apparent again that the removal of the first story infill walls has practically negligible consequences on the seismic capacity of the building.



(a)



(b)



(c) (d)

Fig. 13 Pushover capacity curves of the building along the main directions +X, -X, +Y & -Y

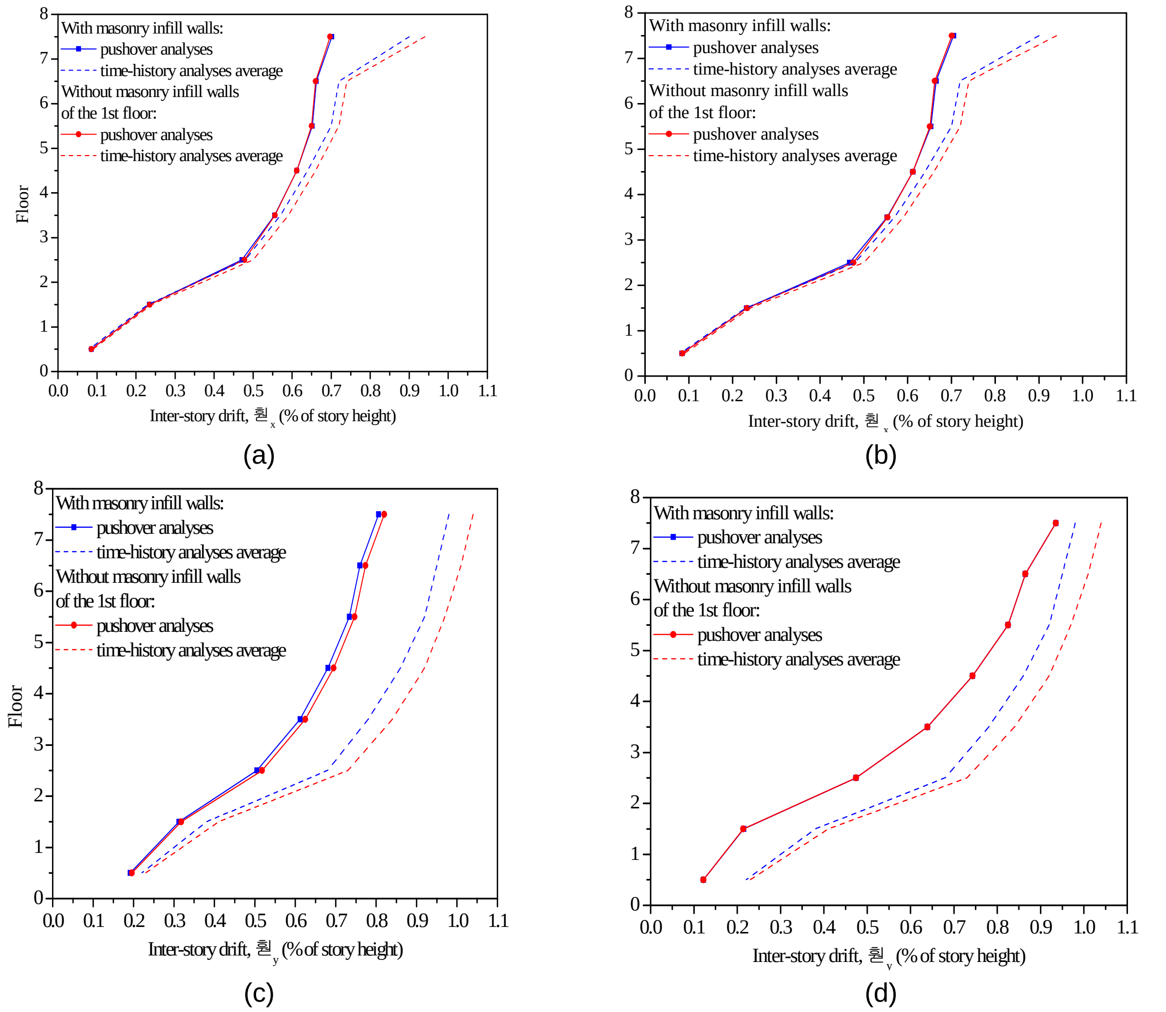


Fig. 14 Interstory drifts by pushover analysis (a) along +X, (b) along -X, (c) along +Y & (d) along -Y

7. CONCLUSIONS

Based on the above and other results not reported herein, the following conclusions have been drawn:

(a) Removal of the first story masonry infill walls has practically no effect on the seismic capacity of the building.
(b) Under current design standards for new buildings, as specified in Eurocode EN 1998-1 (2005), some damage is predicted in the beams connecting to the major shear walls of the building. More significant is the damage expected in the upper stories of columns around the seismic separation of the two building units, namely in columns C7, C14, C21 and C28 (Fig.3).
(c) This suggests that some local strengthening might be required, but not as a result of the planned removal of the first story infill walls.

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