Assessment and Retrofitting of Old Reinforced Concrete Buildings with an Open Ground Story

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SUMMARY:
The inelastic earthquake response of existing reinforced concrete buildings with an open ground story designed according to the old Greek codes is investigated, before and after their seismic strengthening with steel braces restricted to the open ground stories. Based on Part-3 of Eurocode 8 provisions for assessment and retrofitting of buildings, three and five story, symmetric and non-symmetric in plan buildings, are subjected to a set of code compatible synthetic accelerograms, and conclusions are drawn regarding the effectiveness of the strengthening solutions. It is shown that strengthening only the weak ground story, a choice having the substantial advantage of low cost and continued usage of the building during its seismic retrofitting, can remove the inherent weakness without shifting the problem to the stories above, and thus making such buildings at least as strong as those without a weak first story.

Keywords: reinforced concrete buildings, open ground story, seismic evaluation, retrofitting, steel bracing

1. INTRODUCTION

Existing reinforced concrete (RC) buildings with brick infills and open ground stories (pilotis), designed by the Greek codes applicable till 1984, represent a structural type that has suffered most of the heavy damage and collapses during strong earthquakes in Greece in the past 30 years (in the Alcyonides 1981, Kalamata 1986, and Athens 1999 earthquakes) and worldwide (e.g. in the Mexico 1985 and Kocaeli-Izmit 1999 earthquakes). The response of such buildings to earthquake actions is characterized by substantial uncertainty, while their overall behavior is strongly influenced by the response of their open ground stories. Modern building codes for design of new structures include special provisions for buildings with vertical irregularities, one of which is the open ground story. As an example, Eurocode 8 (EC8, 2004) for earthquake resistant design of structures requires an increase in the resistance of the columns in the weak stories, by magnifying their internal forces due to seismic actions in order to prevent formation of a plastic side sway story mechanism.

Unfortunately, this problem was not recognized by older codes and this, combined with other code shortcomings and inadequate construction practices of the past, led to weaker than desired buildings, as numerically documented and witnessed by their performance in recent earthquakes (Antonopoulos and Anagnostopoulos, 2010, Antonopoulos et al, 2008, Repapis et al, 2006). A partial strengthening solution, i.e. a strengthening scheme restricted to the open ground story, that effectively improves the seismic behavior of the building, is apparently a solution that minimizes total cost, while allowing the building to remain operational during the intervention. Among various retrofitting alternatives (Sugano, 1996, Dritsos, 2005, Thermou and Elnashai, 2006), X diagonal steel bracing which increases primarily the lateral strength and secondarily the lateral stiffness of the building is an easy to apply technique, with minimal disturbance (Hemant et al, 2009, Antonopoulos and Anagnostopoulos, 2010). This technique requires a reliable, well detailed and technically sound connection between the steel elements and the existing concrete members.

The main objective of this paper is to examine whether a good retrofitting solution can be found for strengthening only the ground story of "Pilotis" type buildings with steel braces. For this purpose, four buildings, two symmetric having 3 and 5 stories, and two non-symmetric, also with 3 and 5 stories, were selected and were designed according to the old Greek codes (i.e. the old Earthquake Resistant Design Code of 1959 and the old Greek RC Design Code of 1954) so as to represent existing buildings designed and constructed from 1959 till 1984, a year when the Codes changed. Subsequently, these buildings are strengthened by means of suitable X bracing in selected bays of the ground story. Then the seismic behavior of these buildings before and after strengthening is evaluated according to the provisions of Part-3 of Eurocode 8 (EC8-Part 3, 2005) for assessment and retrofitting of buildings, using nonlinear dynamic time history analyses with a set of seven pairs of artificial accelerograms matching closely the code specified design spectrum for Soil Type A and seismic Zone I (EAK, 2003). Finally, conclusions are drawn regarding the effectiveness and feasibility of the proposed partial retrofit solutions.

2. BUILDING DESCRIPTIONS AND STRENGTHENING SOLUTIONS

The buildings analyzed herein are 3 and 5 story RC buildings on pilotis, having brick infill walls in all stories except the ground story. They are space frame structures with two different plan layouts: one symmetric and the other non-symmetric, the latter with an elevator shaft located in a corner of the building and causing bidirectional eccentricity, with e_x=0.15 and e_y=0.19. These eccentricities are the projections on the x and y axes, respectively, of the physical eccentricity, i.e. the distance between the center of mass and an approximate center of stiffness, estimated for all floors according to Stathopoulos and Anagnostopoulos (2005), and normalized by the corresponding maximum building dimension along the x and y axes. Fig. 2.1 shows the typical floor plans of the two layouts, indicating also the bays of the open ground stories where steel braces are placed for strengthening.

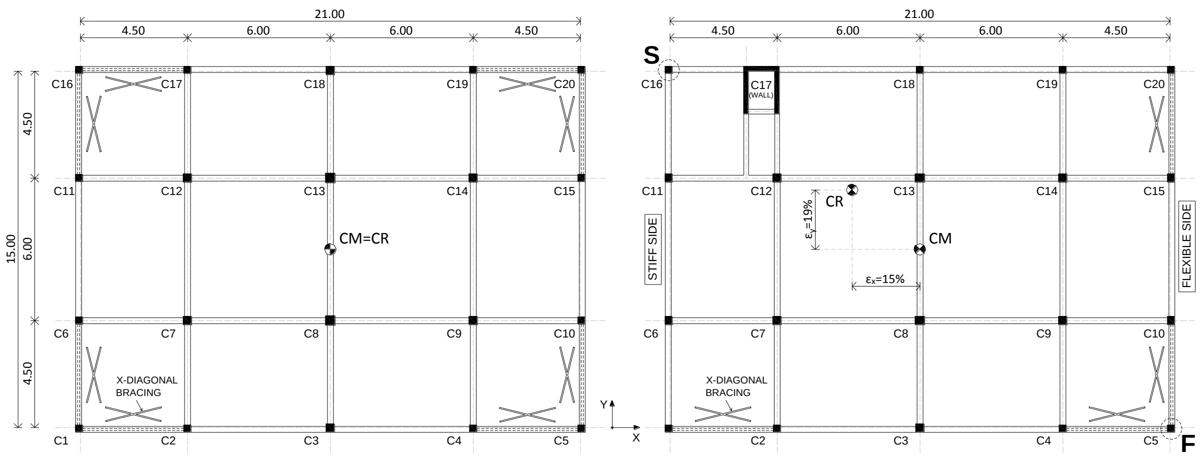

Figure 2.1. Typical layouts for the symmetric (left) and the eccentric (right) 3-story and 5-story buildings

Fig. 2.2 shows the elevation of the 5-story buildings in the y direction, along with a typical detail of X-bracing of the strengthened bays of the open ground story RC frames. Dimensioning of the original buildings was done according to the old Greek codes for reinforced concrete and for earthquake resistant design. The base shear coefficient for seismic actions was taken equal to ε=0.04, corresponding to seismic Zone I and Soil Class A of the old 1959 Code and consequently the design base shear was equal to 4% of the total gravity load G+P (permanent plus live).

Following the common practice of that period, simplified models were used for the calculation of the internal forces while member dimensioning and corresponding design checks followed the allowable stress design method for concrete quality/steel grade B160/St I, both typical construction materials during the sixties and seventies. The longitudinal steel reinforcement ratio for columns ranged between 0.8% and 1.1% of the gross section area, while the transverse reinforcement consisted of smooth steel stirrups, 6mm in diameter, with open, 90˚ hooks, equally spaced at 20cm along the entire member length (non-seismically detailed transverse reinforcement). Longitudinal reinforcement of beams was controlled mainly by gravity loads. For shear reinforcement in beams, 8mm stirrups equally spaced at 25cm was provided everywhere.

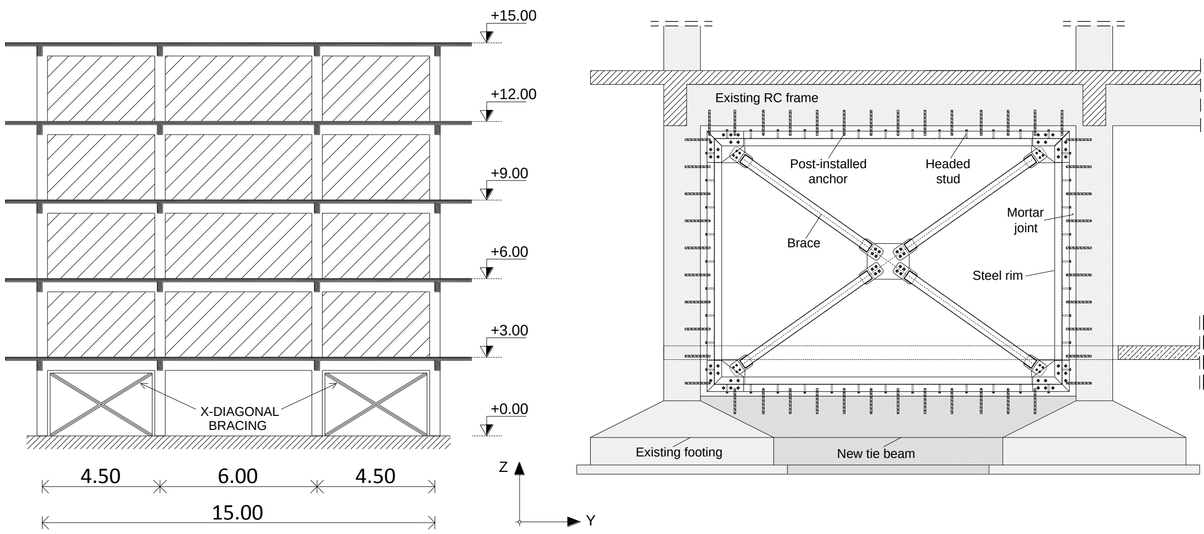

Figure 2.2. Elevation of the 5-story buildings along y direction (left) and typical detail of X-bracing for strengthening the ground story (right)

Based on earlier work reported by Antonopoulos and Anagnostopoulos (2010) and Antonopoulos et al, (2008), all four buildings were strengthened using diagonal steel braces in corner bays of their open ground stories as indicated in Figs. 2.1 and 2.2. The brace sections (listed in Table 2.1) were selected after preliminary analyses with the objective not to overdesign the ground story: a case that would move the structural deficiency to the story above. Thus, the goal was to limit the interstory drift of the ground story to a level comparable to the interstory drift of the story above, and then compare the response of the original and the strengthened building for the selected earthquake action. As will be shown in the subsequent sections, this goal was met and the buildings' performance significantly improved.

Table 2.1. Section profiles of steel bracing

Building	X direction	Y direction
3st-symmetric	SHS – 90/3.6	SHS – 90/3.6
5st-symmetric	CHS – 88.9/3.2	CHS – 88.9/3.2
3st-eccentric	CHS – 108/4.5	CHS – 108/3.6
5st-eccentric	CHS – 88.9/3.2	CHS – 88.9/3.2

3. NON-LINEAR MODELING AND EARTHQUAKE INPUT

Seismic capacity of the buildings before and after strengthening was investigated using nonlinear time history dynamic analyses, based on Part-3 of Eurocode 8 (EC8-Part 3, 2005) for assessment and retrofitting of buildings. Seven pairs of artificial accelerograms were generated using the code by Halldorsson et al (2002). The selected motions comply with the rules of Eurocode 8 (EC8, 2004) for time history representation of the seismic action, i.e. their 5% damped average response spectrum matches the target design spectrum of EAK (2003) for seismic Zone I (PGA=0.16g) and Soil Class A

(Rock), as illustrated in Fig. 3.1.

The nonlinear dynamic analyses of the buildings were carried out using the computer program Ruaumoko 3D (Carr, 2005). Prismatic beam-column frame elements were used to model beams, columns, and the elevator shaft, while brick infill walls and steel braces were modelled using special springs. The effective stiffness of each RC member was taken equal to the secant stiffness at yield according to EC8-Part 3 (2005) based on mean material strengths (f_{cm}=12.8Mpa for concrete, f_{ym}=253Mpa for longitudinal reinforcements and stirrups). Nonlinearity at the two ends of RC members was idealized using one-component plastic hinge models, following the Takeda hysteresis rule with parameters a=0.3 and b=0.0 and a post yield strain hardening ratio equal to p=0.05. Axial force effects on the yield moments of column members were accounted for using appropriate N-M_y-M_z interaction diagrams, obtained from nonlinear fiber cross sectional analysis. Flexibility of joints was neglected but joint dimensions were taken into account through appropriate rigid offsets at member ends.

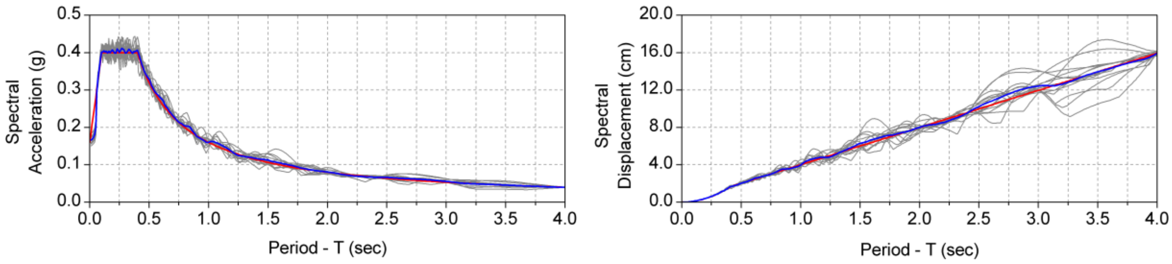

Figure 3.1. Mean response versus target design pseudo-acceleration (left) and displacement (right) spectra for the 14 synthetic accelerograms

Each brick wall panel was modeled with two spring elements, one along each diagonal, with cyclic force–deformation relationships according to Crisafulli and Carr (2007). Based on data by Karantoni (1999), the mean value of the compressive strength of the struts in the direction of the diagonal was calculated equal to f_{wm}=2.3Mpa, with a corresponding strain chosen equal to $\varepsilon_w=0.0015\cdot f_{wm}=0.00345$. A constant width equal to 15% of the clear diagonal length was chosen for infill struts with a thickness equal to 0.20m. For linear modal analysis, where both struts are active, each strut was considered to have half the total horizontal stiffness, i.e. the axial stiffness of each spring was taken equal to $0.5E_wA_w/L_{d,clear}$. A typical value equal to 750 times the mean compressive strength f_{wm} was adopted for the modulus of elasticity E_w of masonry infills (KAN.EPE., 2012). Axial stiffness of the springs (struts) during nonlinear analyses was controlled by the hysteretic rule.

Diagonal steel cross-bracing members were also modelled with spring elements, following bilinear force-deformation relationships. Brace tensile strength was calculated according to Eurocodes 3 and 8 (EC3, 2005 and EC8, 2004) while compressive strength was taken as a fraction of the buckling load (20%), according to the Greek Retrofitting Code (KAN.EPE., 2012). Masses for the dynamic degrees of freedom were calculated from the quasi permanent static combination (G+0.3Q) and considered lumped at the nodes. Rigid diaphragms were assumed at floor levels through appropriate nodal constraints. Rayleigh type viscous damping was used such that 5% modal damping was produced in the lowest two modes of the elastic models.

Table 3.1 summarizes the first three fundamental periods of vibration of the buildings and the effective modal mass ratios along the x and y directions before and after strengthening. The potential for torsional motion is reflected in the effective modal mass ratios. The addition of steel braces at sides opposite to the stiff elevator shaft (see Fig. 2.1) reduces the eccentricity and the resulting torsion, as can be inferred by comparing the effective modal mass ratios of the non symmetric buildings before and after strengthening (original vs braced).

Table 3.1. Modal data for the first 3 modes of the original and the braced buildings

Building	Mode	Period T (sec)		Modal mass M^*_x (%)		Modal mass M^*_y (%)	
		Original	Braced	Original	Braced	Original	Braced
3st-symmetric	1	0.766	0.541	-	-	94.0	78.0
	2	0.746	0.522	91.0	77.0	-	-
	3	0.694	0.404	4.0	2.0	-	-
5st-symmetric	1	1.030	0.904	-	-	87.0	79.0
	2	0.995	0.874	85.0	78.0	-	-
	3	0.866	0.697	3.0	2.0	-	-
3st-eccentric	1	0.726	0.550	34.0	11.0	31.0	69.0
	2	0.602	0.519	41.0	71.0	47.0	11.0
	3	0.493	0.423	15.0	-	10.0	1.0
5st-eccentric	1	0.978	0.884	32.0	12.0	37.0	66.0
	2	0.884	0.848	39.0	64.0	41.0	13.0
	3	0.732	0.693	12.0	4.0	4.0	-

4. NON-LINEAR TIME HISTORY ANALYSES RESULTS

Each building was analyzed for the selected motion pairs and peak response quantities were calculated during the analyses and also through step by step post processing of each analysis set. Subsequently, these peaks were averaged over the 2x7=14 analyses sets. Seismic performance of the buildings was evaluated according to EC8-Part 3 (2005). The design seismic action for which buildings were analyzed corresponds to the Limit State (LS) of Significant Damage (SD). Average values of the absolute maximum interstory drifts of the 3 and 5 story symmetric and eccentric buildings are shown in Figs. 4.1 to 4.4, respectively. Looking at the response of the original 3-story building (dashed line in Fig. 4.1), a clear soft story behavior is apparent, because the largest portion of the total displacements of the building in both x and y directions is concentrated at the open ground floor. Corresponding results of the 5-story symmetric original building show a less obvious soft ground story behavior, but also in this case the ground story develops the largest interstory drifts compared to the stories above. Comparing interstory drifts in Figs. 4.1 and 4.2 between the original and the strengthened buildings, we observe a significant reduction of the ground story displacements, in both directions of both buildings, with little change in the stories above.

Regarding the 3-story and 5-story eccentric layouts, interstory drifts are given for the “stiff” and “flexible” edges of the buildings at points “s” and “f”, respectively (see corresponding layout in Fig. 2.1). Overall, the behaviour in these cases is governed by torsional response, and any soft story effects are apparent on the “flexible” sides of the buildings, i.e. the sides forming a corner diagonally opposite the elevator shaft. This is clearly seen in the response of the original 3-story eccentric building (see Fig. 4.3) but it is not as obvious in the response of the 5-story eccentric building (see Fig. 4.4), as was also true for the symmetric cases. In cases of eccentric open ground stories, the strengthening scheme should aim not only at strengthening the soft story but also at reducing eccentricities and the consequent torsional response. This is what happened in the case of the 3-story eccentric building where, after the addition of steel bracing, both of these negative response factors were minimized. On the other hand, in the case of the 5-story eccentric building, the beneficial effects of the ground story bracing are clear only where they are needed the most, i.e. in the ground story, while in the upper stories, where the original eccentricities have not been affected, the torsional response is still apparent.

Member verifications were carried out for all components (beams, columns and walls), both for flexure under bending moments with axial load (ductile behavior) and for shear force (brittle behavior). As stated earlier and also reported by Antonopoulos and Anagnostopoulos (2010), strengthening of the ground story beyond a certain limit will shift the problem to the stories above, whose interstory drifts will increase, especially in the first story above. This shift gradually disappears as we move to higher stories. For this reason an upper limit must be found for the ground story strengthening, as it has been done here, and member checks must be repeated for all structural members.

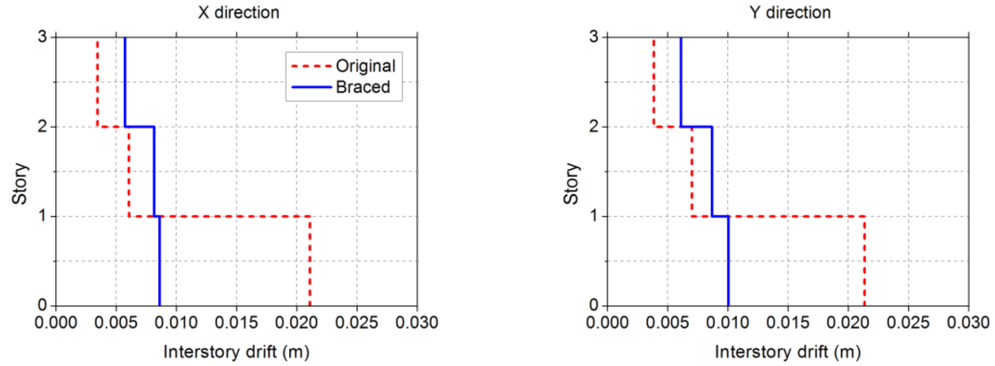

Figure 4.1. 3-story symmetric building: average values of interstory drifts along X and Y direction

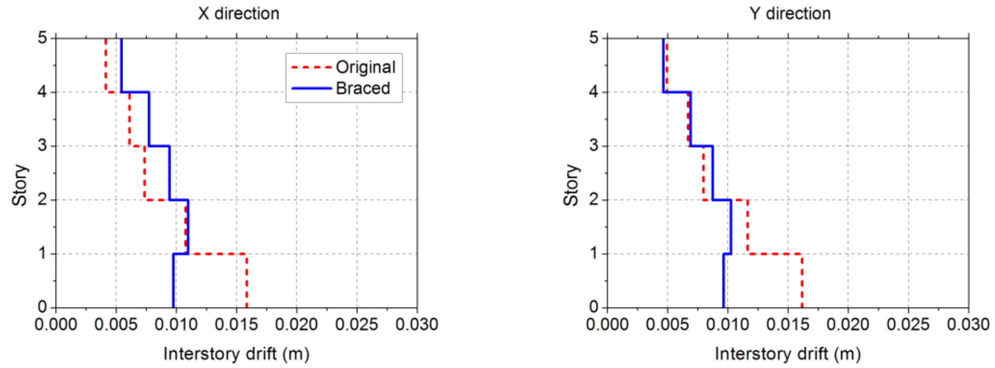

Figure 4.2. 5-story symmetric building: average values of interstory drifts along X and Y direction

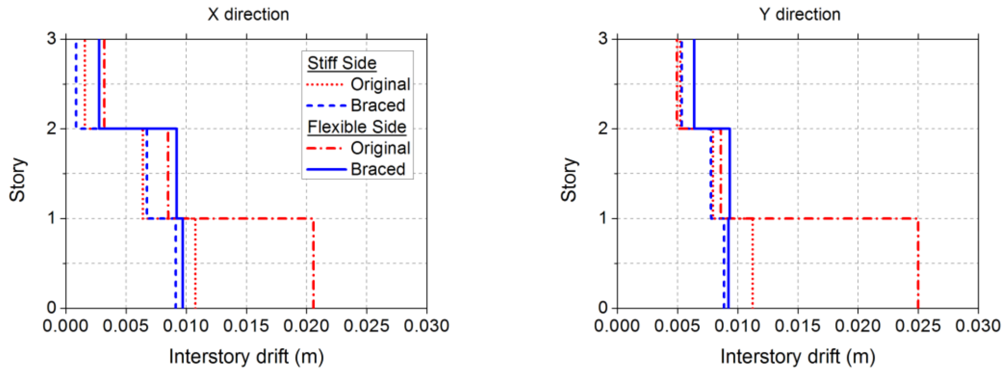

Figure 4.3. 3-story eccentric building: average values of interstory drifts along X and Y direction

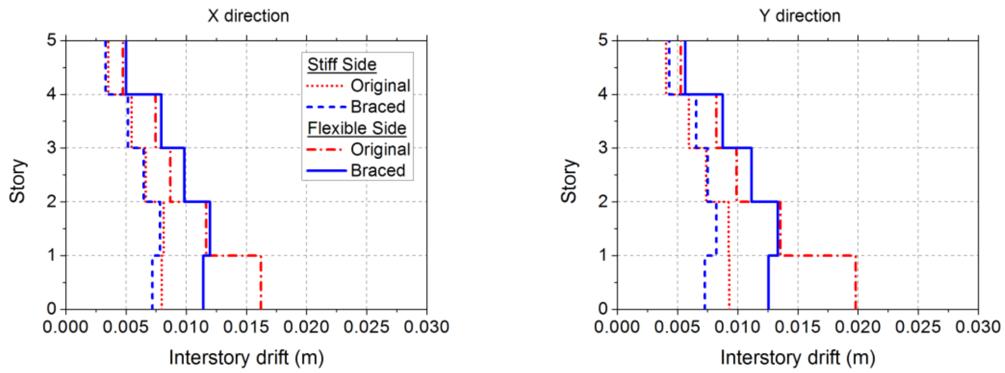

Figure 4.4. 5-story eccentric building: average values of interstory drifts along X and Y direction

Figures 4.5 to 4.8 summarize basic quantities for the overall stability of the buildings, i.e. shear Demand/Capacity (D/C) checks of the ground story columns, shear D/C checks of the first story columns as well as bending D/C checks of plastic hinge rotations of the ground story columns. These bending checks represent demand to capacity ratios of the maximum plastic hinge rotations to the instantaneous (due to variation of the axial loads) plastic rotation capacity of members with smooth longitudinal bars, without detailing for earthquake resistance, based on mean material strengths, calculated according to EC8-Part 3 (2005). Strength and deformability modification due to lap-splicing of the column reinforcement at floor levels was ignored. In each analysis step, D/C ratios of the plastic rotations were calculated separately for each of the two principal axes of bending (y and z) at both member ends (i and j), as well as according to the following gross rule of instantaneous combination of the two plastic rotations along principal axes y and z at the cross section level:

$$D/C_{\theta_{pl}} = \sqrt{\left(\frac{\theta_{pl,y}}{\theta_{um,EC8}^{pl,y}}\right)^2 + \left(\frac{\theta_{pl,z}}{\theta_{um,EC8}^{pl,z}}\right)^2} \tag{4.1}$$

In calculations of the mean values, plastic rotations were considered equal to zero for members that remained elastic during response. Shear demand to capacity checks are ratios of the applied shear force, to the instantaneous cyclic shear resistance V_R of the members in each of the two principal axes. In this calculation, mean material strengths were additionally divided by the partial material factors, according to EC8-Part 3 (2005). The contribution of stirrups to the calculations of cyclic shear resistance was reduced to half its calculated value, due to non-seismically detailed open stirrups (see, Biskinis et al, 2004).

The substantial reduction of the ground story displacements resulted in lowering the corresponding maximum column shears, so that the several D/C ratios that exceeded 1.0 in the original building, indicative of high risk for failure, now were reduced to values below 1.0, as may be seen in Fig. 4.5 and 4.6. The expected increase of the seismic demands in the story just above the ground story, after the strengthening, may be seen in the first story column D/C ratios, but their values are still on the safe side, below 1.0. Bending checks quantified by the plastic hinge rotation D/C ratios, are less critical. Note that the values of the plastic rotation capacities in the denominator of equation (4.1) are those corresponding to the Limit State of Near Collapse (NC). EC8-Part 3 (2005) specifies that the chord rotation capacity, corresponding to the LS of Significant Damage, may be assumed as 3/4 of the value corresponding to chord rotation at the LS of Near Collapse. In terms of maximum plastic hinge rotations, the plastic rotation corresponding to the LS of Significant Damage is approximately 1/2 of that corresponding to the LS of Near Collapse if a maximum available ductility factor equal to 2.00 is considered as in the case of columns examined herein. In other words, column members with bending D/C ratios above 0.5 have already exceeded the LS of Significant Damage under which the performance of the buildings is evaluated, and thus these members may be considered as failing in bending.

As in the case of the symmetric buildings, steel bracing reduces significantly the potential for shear failures in the ground story columns. However, as may be seen in Figs. 4.7 and 4.8, one wall element still has a shear D/C ratio above 1.0, which means that additional measures may be required locally because, as mentioned earlier, an upper limit exists in strengthening the ground story without significantly overloading the story above. Regarding the masonry infills, which inevitably play their role on the global seismic response of the buildings, a damage index equal to the ratio of the maximum axial deformation to the deformation at maximum strength of each infill was selected as a key value to measure their damage, in terms of deformations. After calculating this index for the two infill struts (springs) of each panel separately, average values among all infills in each direction were calculated as global story infill damage indices. Figs. 4.9 and 4.10 show average values of this index in the x and y directions before and after seismic strengthening of the symmetric and the eccentric buildings, respectively. Conventionally, values greater than 1.0 correspond to infills that have reached their maximum available strength and have started to degrade.

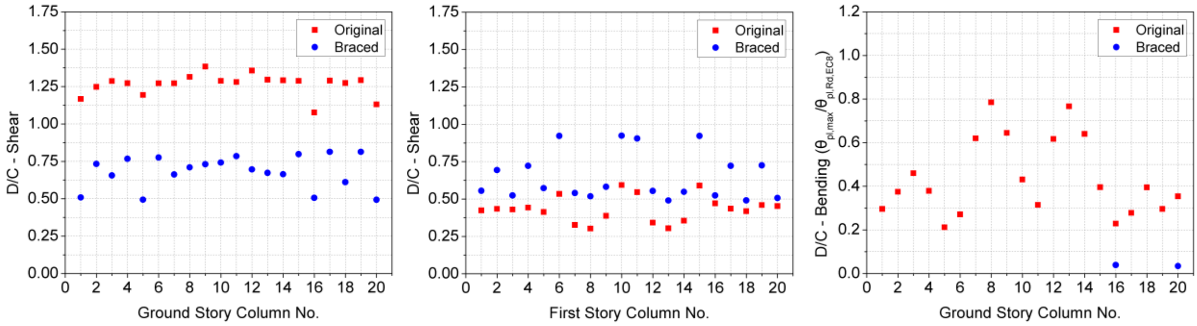

Figure 4.5. 3-story symmetric building: Shear D/C ratios of ground story columns (left), shear D/C ratios of first story columns (center) and bending D/C ratios of ground story columns (right)

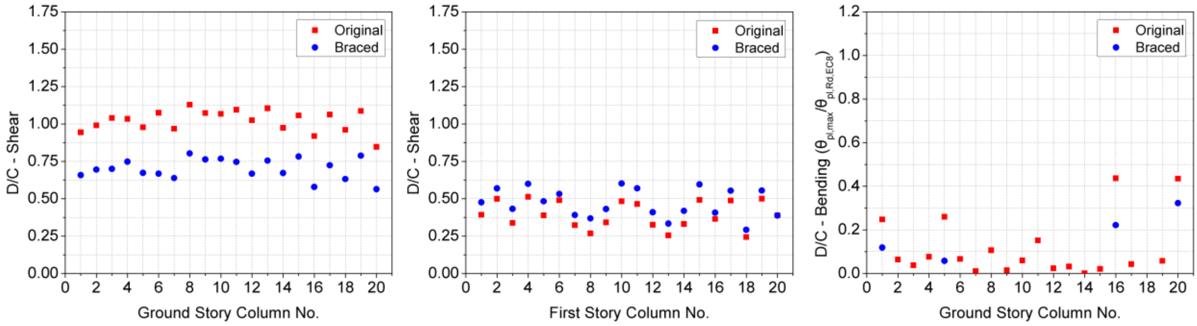

Figure 4.6. 5-story symmetric building: Shear D/C ratios of ground story columns (left), shear D/C ratios of first story columns (center) and bending D/C ratios of ground story columns (right)

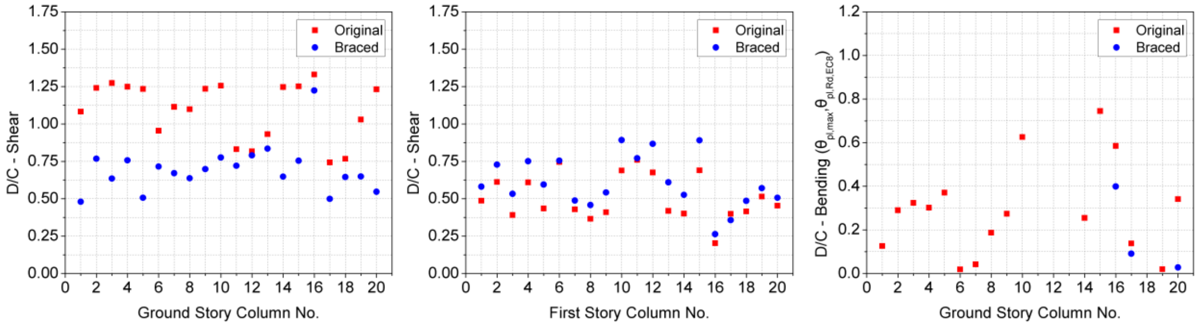

Figure 4.7. 3-story eccentric building: Shear D/C ratios of ground story columns (left), shear D/C ratios of first story columns (center) and bending D/C ratios of ground story columns (right)

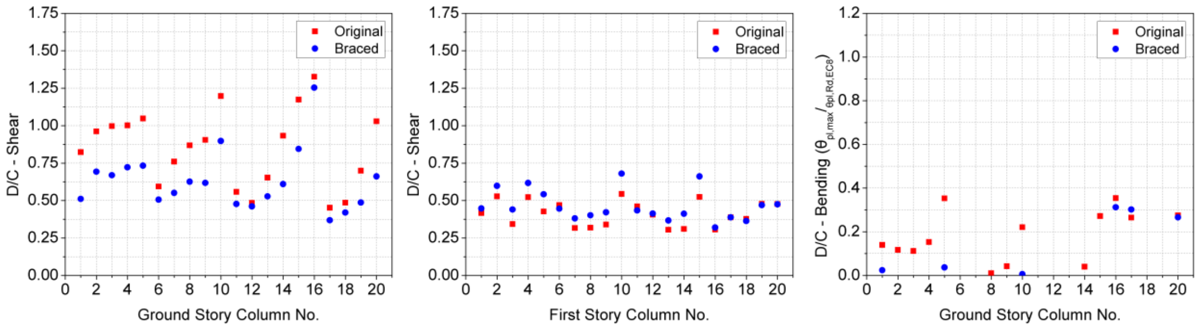

Figure 4.8. 5-story eccentric building: Shear D/C ratios of ground story columns (left), shear D/C ratios of first story columns (center) and bending D/C ratios of ground story columns (right)

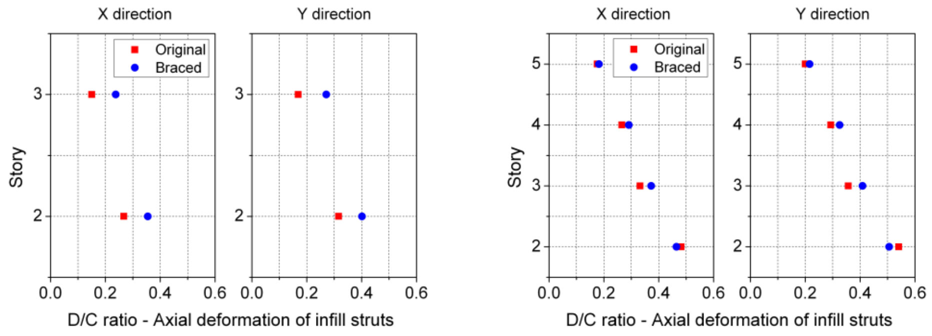

Figure 4.9. D/C ratios in infills of the 3st-symmetric building (left) and 5st-symmetric building (right)

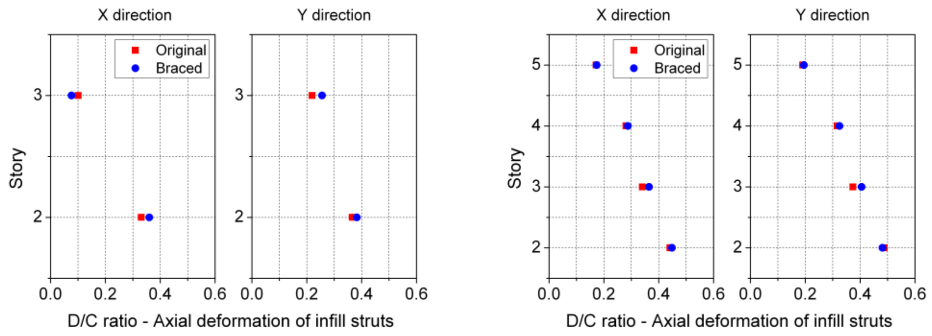

Figure 4.10. D/C ratios in infills of the 3st-eccentric building (left) and 5st-eccentric building (right)

These results indicate no significant infill damage in the original buildings – where most of the damage is concentrated at the ground story – and some insignificant increase of the D/C ratios as a result of strengthening. Average values of maximum ductility factors in brace elements for the four cases of symmetric and eccentric buildings are listed in Table 4.1. They are relatively low, implying that the braces can sustain even larger axial deformations in some future earthquake, stronger than a design event.

Table 4.1. Average values of the maximum ductility factors of the brace elements (tension only)

Building	X direction	Y direction
3st-symmetric	1.49	1.41
5st-symmetric	1.78	1.76
3st-eccentric	1.55	1.48
5st-eccentric	1.86	2.06

5. CONCLUDING REMARKS

The work reported herein addresses the problem of strengthening the most vulnerable class of existing RC buildings in Greece, namely buildings with an open ground story (pilotis), designed and built under old Greek codes and practices, and which have performed very poorly during earthquakes of the last 30 years. The present paper examined the feasibility of partial strengthening of such buildings, aiming at reducing their vulnerability due to the weak first story and lowering it to a level comparable to that of regular buildings i.e. having sufficient infill walls in the ground story. The partial strengthening by intervening only in the open ground story, as opposed to a complete strengthening to comply with current standards for new buildings, is perhaps the only retrofitting possibility that might be acceptable by the owners of such buildings, due to: (a) low cost of intervention and (b) continued usage of the building during the retrofitting work. Based on inelastic, dynamic earthquake response analyses of two symmetric such buildings with 3 and 5 stories and two eccentric such buildings also with 3 and 5 stories, their vulnerability due to the weak ground story was first confirmed.

Subsequently, these buildings were strengthened with steel braces placed in appropriately-selected bays of the ground story, and their performance under the same earthquake set was examined. Both the symmetric and non-symmetric cases showed greatly improved response, which met the set objective of removing the ground story weakness without moving the problem to higher stories. Note also that with the selected bracing locations in the case of non-symmetric buildings, it was possible to drastically reduce the ground story eccentricity, and through that, the undesirable torsional response of the building. It is believed that the proposed retrofitting scheme, which is perhaps the only feasible way of strengthening a building with open ground story that would be acceptable to its owners, could indeed save such a building from collapse or very heavy damage in a future catastrophic earthquake.

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