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Seismic evaluation and upgrading of RC buildings with weak open ground stories

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Abstract. 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. The seismic performance evaluation is based on Part 3 of Eurocode 8 for assessment and retrofitting of buildings. Three and five-story, symmetric and non-symmetric buildings are subjected to a set of seven pairs of synthetic accelerograms, compatible with the design spectrum, and conclusions are drawn regarding the effectiveness of the strengthening solutions. Seismic behavior of the selected models confirms results of previous work regarding the insufficient capacity of the open ground stories for design level earthquakes. It is also 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. This partial strengthening is possible for symmetric as well as eccentric buildings, in which torsion plays a further detrimental role.

Keywords: reinforced concrete buildings; open ground story; inelastic earthquake response; seismic evaluation; seismic strengthening; 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, and a weak story is one of them. As an example, Eurocode 8 (CEN 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

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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), steel bracing provides a suitable, easy to apply choice with minimal disturbance (Antonopoulos and Anagnostopoulos 2010).

Use of braces in selected bays of existing RC buildings is effective for global strengthening, provided that a reliable, well detailed and technically sound connection between the steel elements and the existing concrete members is ensured. This has been recognized both experimentally (e.g. Sugano and Fujimura 1980, Tagawa *et al*. 1992, Maheri and Sahebi 1997) and numerically (e.g. Yamamoto and Umemura 1992). Architectural constraints related to the strengthening schemes can be addressed through alternative choices of bays to be braced. In this manner, a strengthening solution could be achieved by using X-, V-, or inverted-V bracing, like those examined, e.g., by Jain (1985), Bush *et al*. (1991), Pincheira and Jirsa (1995), Gilmore *et al*. (1996), or Badoux and Jirsa (1990). Alternative retrofitting methods of non ductile RC frames include use of eccentric steel braces with vertical shear links as energy dissipation elements (see, e.g., Ghobarah and Elfath 2001, Perera *et al*. 2004, Ferraioli *et al*. 2006), use of post-tensioned externally anchored steel rods, attached to the perimeter of buildings (Pincheira and Jirsa 1992) or even optimal placement of viscous dampers throughout the buildings (e.g. Fujita *et al*. 2010, Lavan and Levy 2010). Among these alternatives, diagonal X bracing is the most common technique, providing considerable increase primarily in lateral strength and secondarily in lateral stiffness of the building (Hemant *et al*. 2009).

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, the year when the Code 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 for assessment and retrofitting of buildings, using nonlinear dynamic time history analyses for a set of seven artificial accelerograms matching closely the code specified design spectrum for seismic Zone I. Finally, conclusions are drawn regarding the effectiveness and feasibility of the proposed partial retrofit solution.

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. Figs. 1 and 2 show the typical floor plans of the two layouts and corresponding elevations, indicating also the bays where steel braces are placed for strengthening.

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 selected equal to $\varepsilon = 0.04$, corresponding to seismic Zone I and Soil Class A of the old 1959 Code and thus, the design base shear was taken equal to 4% of the total gravity load G+P (permanent

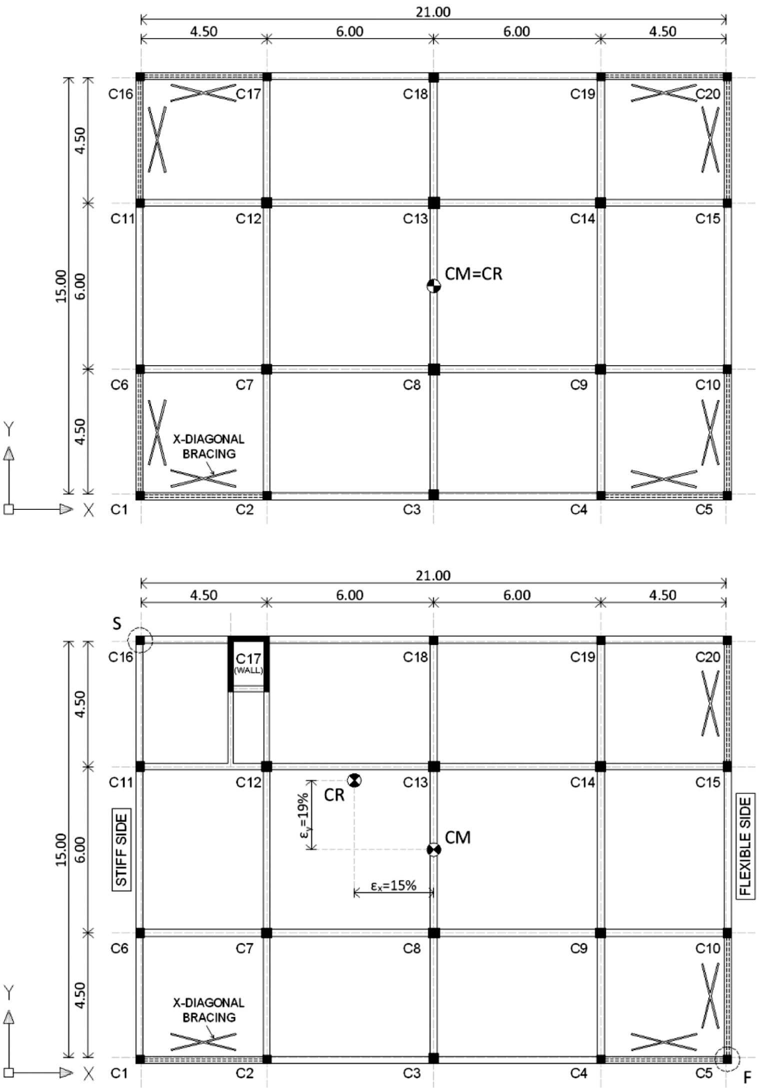

Fig. 1 Typical layouts for the symmetric (top) and eccentric (bottom) 3 and 5 story buildings

plus live). 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.

Following the common practice of that period, simplified models were used for the calculation of the internal forces and the dimensioning of the members: continuous, simply supported beams were assumed for gravity loads, while column and shear wall seismic forces were determined story by story, assuming no joint rotations (i.e., a shear beam model).

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, 6 mm in diameter, with open, 90° hooks, equally spaced at 20 cm 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, 8 mm stirrups equally spaced at 25 cm

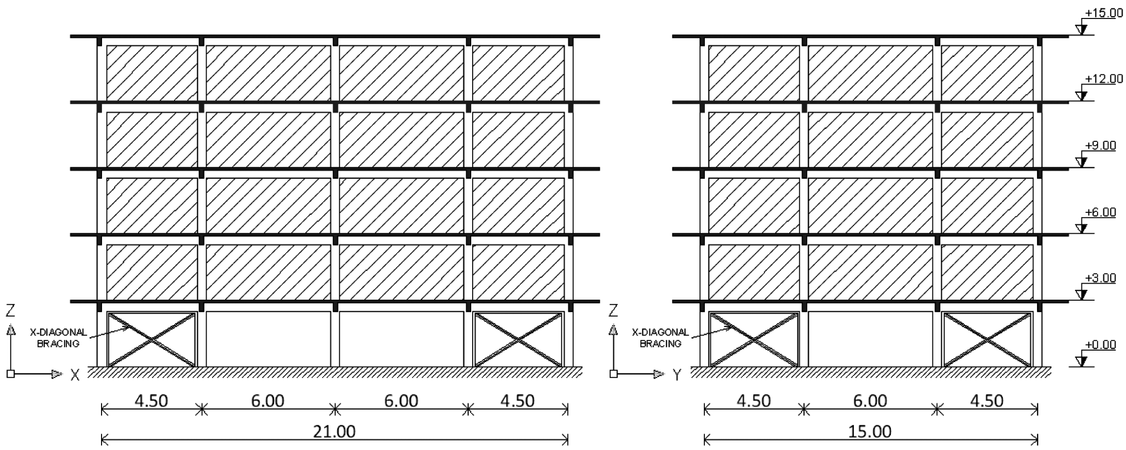

Fig. 2 Elevations of the 5-story buildings along x & y directions

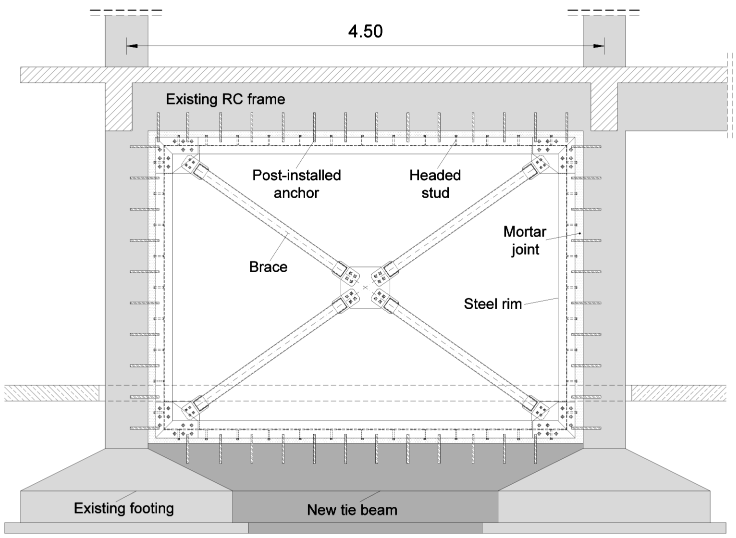

Fig. 3 Typical detail of X-bracing for strengthening the ground story

was provided everywhere. Dimensions and reinforcement for the columns of the ground story level are summarized for all buildings in Appendix-A.

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. 1 and 2. A typical detail of *X*-bracing for strengthening the ground story is illustrated in Fig. 3.

The brace sections (listed in Appendix A) 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 was significantly improved.

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 (CEN 2005b) 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 (CEN 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=0.16 g) and Soil Class A (Rock), as illustrated in Fig. 4.

Modelling and analyses of the buildings were carried out using Ruaumoko 3D (Carr 2005). Prismatic beam elements were used to model beams, columns and the elevator shaft, while brick infill walls and steel bracing were modelled using special spring elements. The effective stiffness of each RC member was taken equal to the secant stiffness at yield (CEN 2005b) based on mean material strengths. This means that the mean value of $M_y L_v / 3\theta_y$ for positive and negative bending at the two ends of each member was calculated, assuming for axial loads in columns and walls those from the quasi permanent gravity load combination G+0.3Q (G and Q are the gravity and live loads, respectively). A mean compressive strength equal to f_{cm}=12.8 Mpa was selected for B160 concrete ($f_{cm} \cong 0.80 \cdot f_{c,cube,0.2x0.2} \cong 0.80 \cdot 160$ Kgr/cm^2), while a mean yield strength equal to f_{ym}=253 Mpa was assumed for longitudinal steel reinforcement and stirrups. Shear span length L_v was taken to be constant and equal to half the member length for beams and columns, while for elevator shaft elements, it was assumed equal to half the distance between the bottom section of an element in a

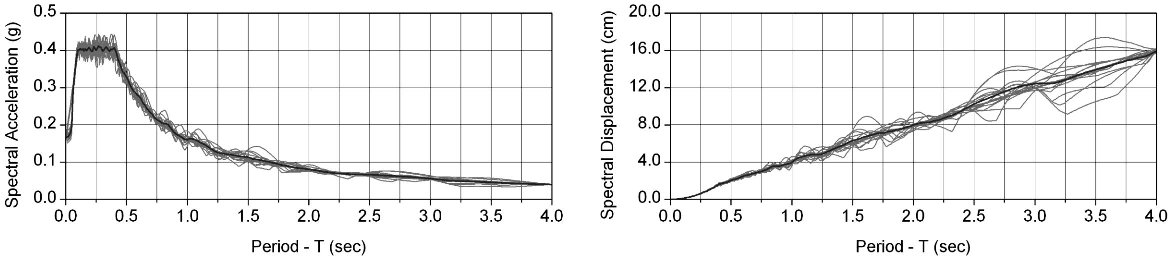

Fig. 4 Mean response versus target code design spectra for the 14 ground accelerograms

story and the element section at the top of the building. Chord rotations θ_y were calculated according to CEN (2005b).

One-component plastic hinge models were chosen to idealize nonlinearity at the two ends of RC members, according to the Takeda hysteresis rule with parameters a=0.3, b=0.0 and a strain hardening ratio (post yield slope) equal to p=0.05. Axial force effects on the yield moments of columns were accounted for using N-M_y-M_z interaction diagrams obtained from nonlinear fiber element analyses of the various cross sections used. Flexibility of joints was neglected but joint dimensions were taken into account through appropriate rigid offsets at member ends. Each brick wall panel was modeled with two spring elements, one along each diagonal, with cyclic force – deformation relationships according to Crisafulli (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}=2.3 Mpa, with a corresponding strain chosen equal to ε_w=0.00345 (=0.0015$\cdot f_{wm}$). A constant width equal to 15% of the clear diagonal length was chosen for infill struts with a thickness equal to 0.20 m. 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

Table 1 Modal data for the first 3 modes of the original and braced buildings

Building	Mode	T (sec)	M_x (%)	M_y (%)
3-story symmetric original	1	0.766	-	94.0
	2	0.746	91.0	-
	3	0.694	4.0	-
3-story symmetric braced	1	0.541	-	78.0
	2	0.522	77.0	-
	3	0.404	2.0	-
5-story symmetric original	1	1.030	-	87.0
	2	0.995	85.0	-
	3	0.866	3.0	-
5-story symmetric braced	1	0.904	-	79.0
	2	0.874	78.0	-
	3	0.697	2.0	-
3-story non-symmetric original	1	0.726	34.0	31.0
	2	0.602	41.0	47.0
	3	0.493	15.0	10.0
3-story non-symmetric braced	1	0.550	11.0	69.0
	2	0.519	71.0	11.0
	3	0.423	-	1.0
5-story non-symmetric original	1	0.978	32.0	37.0
	2	0.884	39.0	41.0
	3	0.732	12.0	4.0
5-story non-symmetric braced	1	0.884	12.0	66.0
	2	0.848	64.0	13.0
	3	0.693	4.0	-

equal to 0.5 $E_w A_w / L_{d,clear}$. A typical value equal to 750 times the mean compressive strength f_{wm} was adopted for the modulus of elasticity E_w. 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. The tensile strength of the braces was taken equal to their yield strength and calculated according to Eurocode 3 (CEN 2005a), while yielding in compression, as ASCE (2007) recommends, was taken equal to a fraction, 1/5, of their buckling load, which was also calculated according to CEN (2005a). 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 two lowest modes of the elastic models. Table 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 the stiff elevator shaft (see Fig. 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.

4. Nonlinear 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 2×7=14 analyses sets. As already mentioned, seismic behavior of the buildings was evaluated according to CEN (2005b). This means that member verification was 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). The level of the design seismic action for which the buildings were analyzed corresponds to the Limit State (LS) of Significant Damage (SD), according to CEN (2005b).

Key parameters for seismic capacity assessment before and after strengthening are the mean values over all motions of the maxima of the following response quantities:

1. Maximum (total) absolute displacements along building height.
2. Maximum absolute interstory drifts (relative story displacements).
3. Ductility demands in beams and columns, defined as

$$\mu_\theta = 1 + \frac{\theta_{pl}}{\theta_y} \tag{1}$$

where, θ_{pl} is the maximum plastic hinge rotation at either end of the members, and θ_y is the corresponding chord rotation at yield, calculated according to CEN (2005b). For consistency with the mathematical modeling and the effective stiffnesses of the members, yield rotations θ_y were assumed constant, and equal to those initially calculated under the action of the quasi permanent gravity loads.

4. Demand to capacity (D/C) 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 CEN (2005b). 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

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

In calculations of the mean values, plastic rotations and ductility factors were considered equal to zero and one, respectively, for members that remained elastic during response.

5. Demand to capacity ratios of the applied shear force, to the instantaneous cyclic shear resistance V_R. In this calculation, mean material strengths were additionally divided by the partial material factors, according to CEN (2005b). The contribution of stirrups to the calculations of cyclic shear resistance was reduced to half its calculated value due to open stirrups (see, Biskinis *et al.* 2004).

Distributions of the maximum absolute displacements over the height as well as maximum relative displacements (interstory drifts) of the symmetric 3 and 5 story buildings are shown in Figs. 5 and 6, respectively. Looking at the response of the original 3-story building (dashed line in Fig. 5), 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 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 total displacements and interstory drifts in Figs. 5 and 6 of 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.

Even better are the results of the member checks. This can be seen in Figs. 7 and 8 that summarize basic quantities for the overall stability of the buildings, i.e., shear D/C checks of the ground and first story columns, maximum ductility demands and bending D/C checks of plastic hinge rotations of the ground story columns. 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.

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 illustrated in Figs. 7(a) and 8(a). In Figs. 7(b) and 8(b) we can see the expected increase of these ratios in the story just above the ground story, but their values are still below 1.0. Bending checks, quantified by the ductility factors and the plastic hinge rotation D/C ratios, are less critical.

Note that the values of the plastic rotation capacities in the denominator of Eq. (2) are those

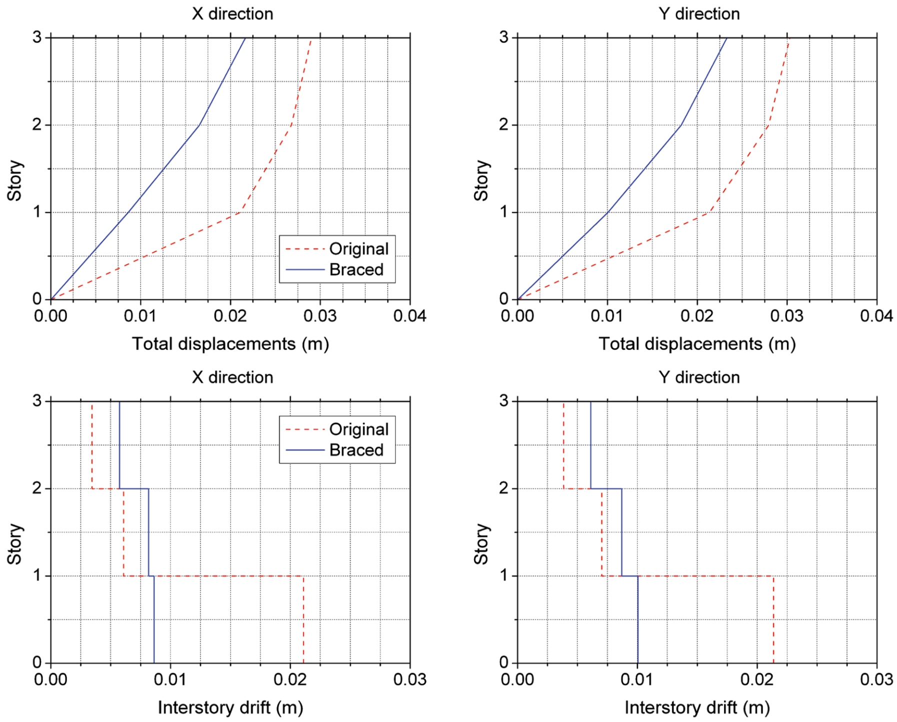

Fig. 5 3-story symmetric building: Total displacements and interstory drifts

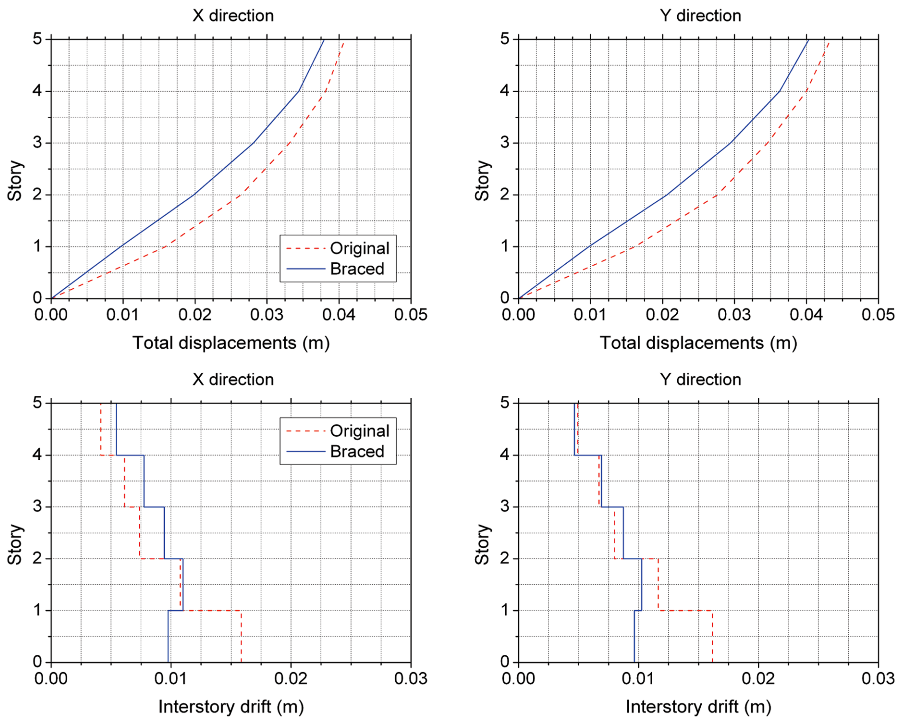

Fig. 6 5-story symmetric building: Total displacements and interstory drifts

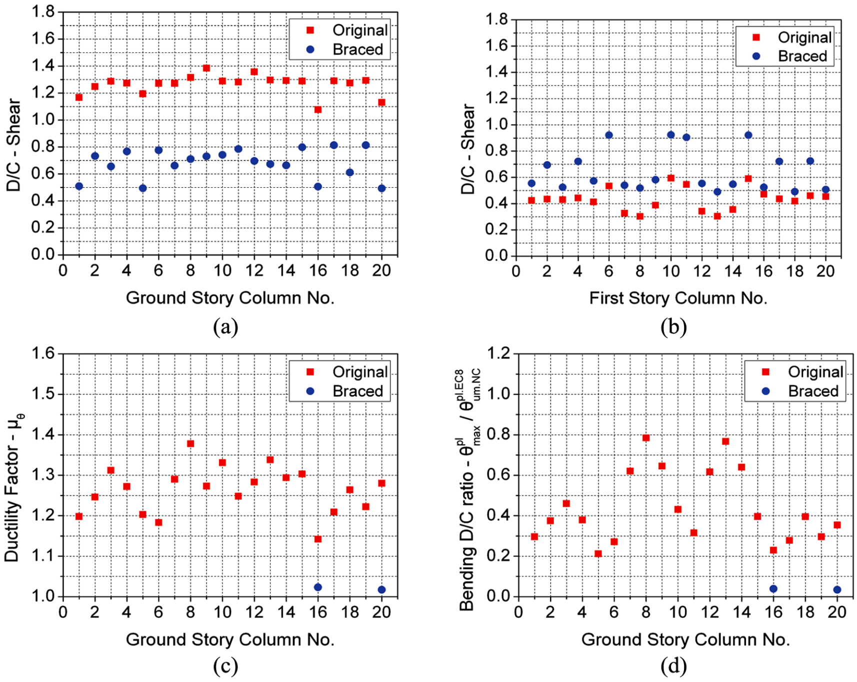

Fig. 7 3-story symmetric building: Shear *D*/*C* ratios in (a) ground story and (b) first story columns, (c) Rotational ductility μ_θ and (d) bending *D*/*C* ratios in ground story columns

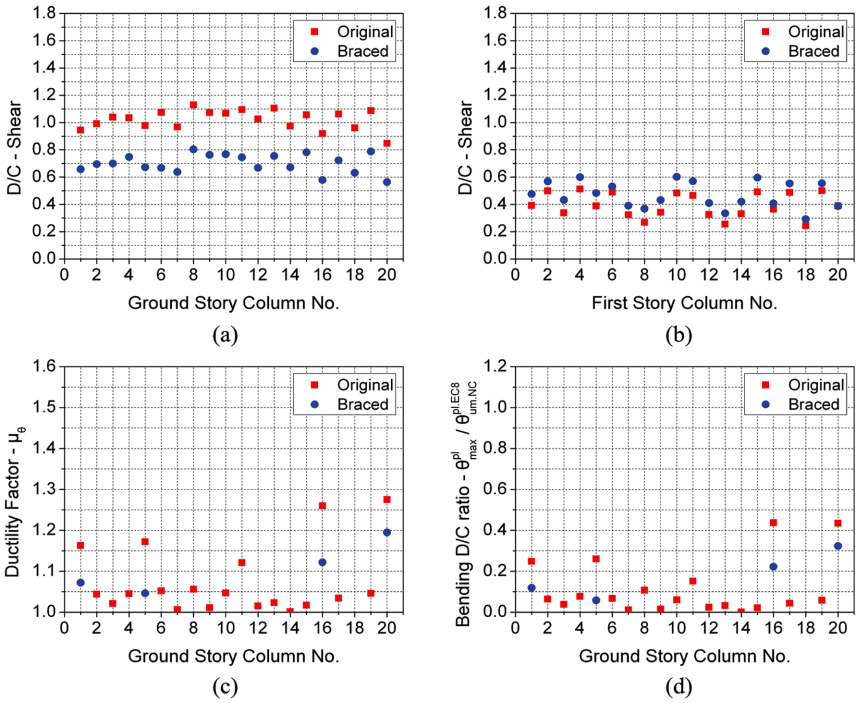

Fig. 8 5-story symmetric building: Shear *D*/*C* ratios in (a) ground story and (b) first story columns, (c) rotational ductility μ_θ and (d) bending *D*/*C* ratios in ground story columns corresponding to the Limit State of Near Collapse (NC). CEN (2005b) specifies that the chord rotation capacity, 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 demand to capacity 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.

Looking at the ductility demands, Figs. 7(c) and 8(c), it is worth noting that the high D/C ratios in shear or bending, indicative of potential failure, occur at relatively low values of ductility factors as would be expected for old, non-seismically detailed columns.

Displacement results for the 3 and 5 story eccentric buildings are shown in Figs. 9 and 10, respectively. They are given for the “stiff” and “flexible” edges of the buildings at points “*s*” and “*f*”, respectively (see Fig. 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 but it is not as obvious in the response of the 5-story eccentric building, 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 subsequent 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.

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. 11(a) and 12(a), 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. Fig. 13 shows average values of this index in the x and y directions before and after seismic strengthening.

Conventionally, values greater than 1.0 correspond to infills that have reached their maximum available strength and have started to degrade. 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 2. They are relatively low, implying that the braces can sustain even larger axial deformations in some future earthquake, stronger than a design event.

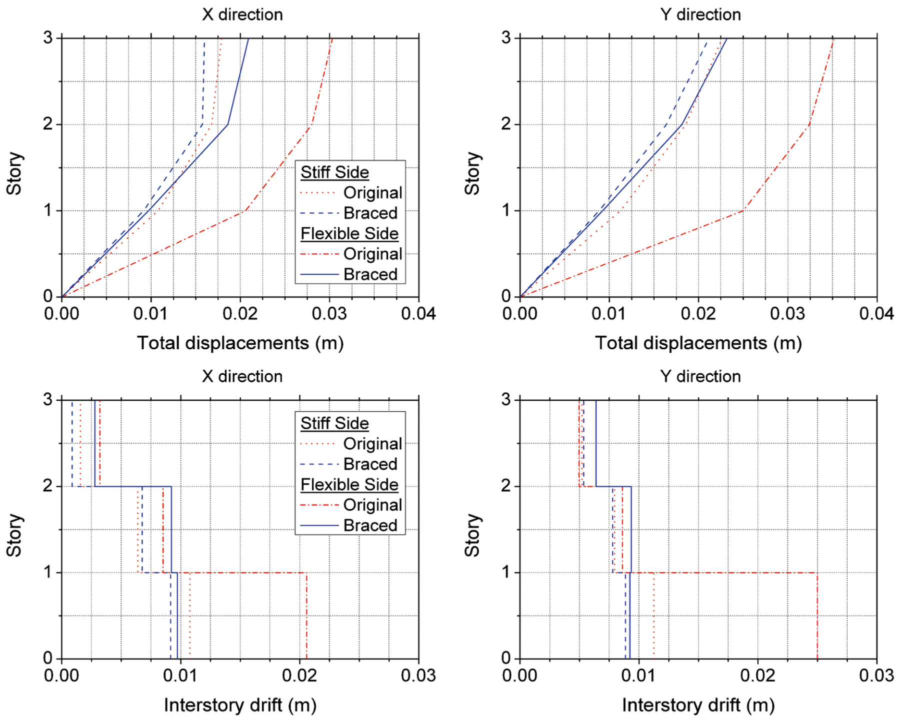

Fig. 9 3-story eccentric building: Total displacements and interstory drifts

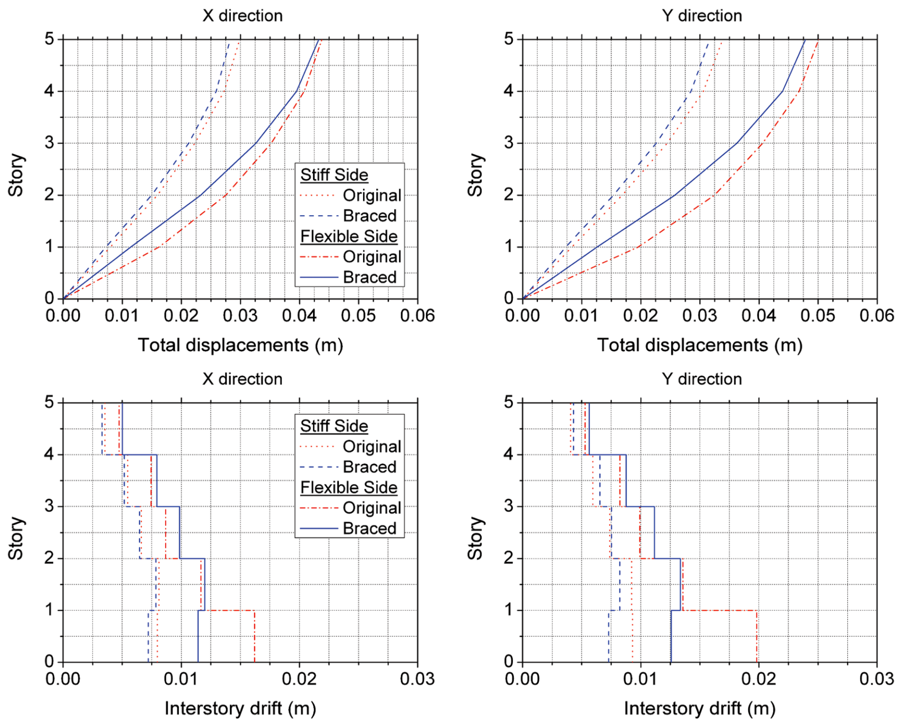

Fig. 10 5-story eccentric building: Total displacements and interstory drifts

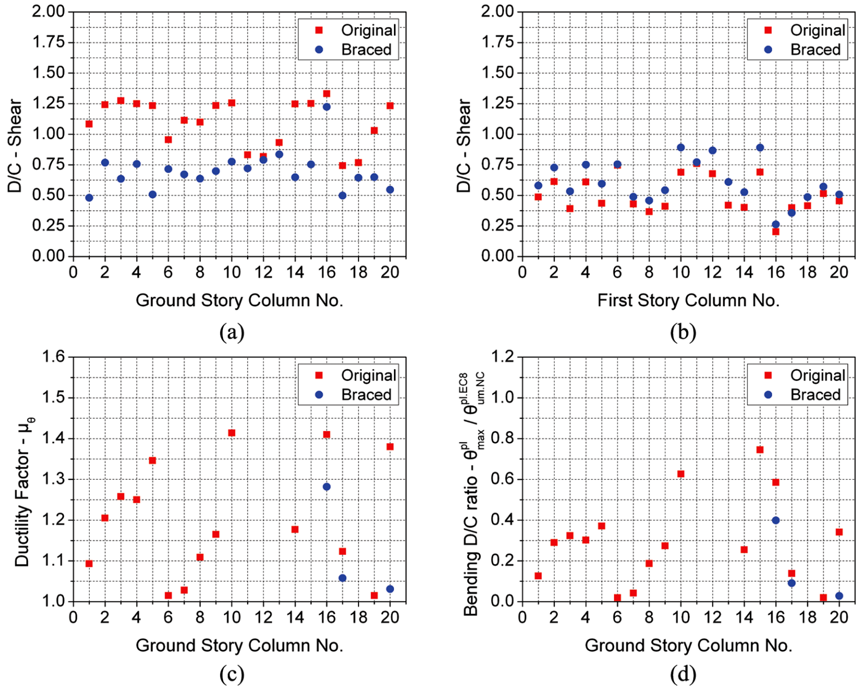

Fig. 11 3-story eccentric building: Shear *D*/*C* ratios in (a) ground story and (b) first story columns, (c) rotational ductility μ_θ and (d) bending *D*/*C* ratios in ground story columns

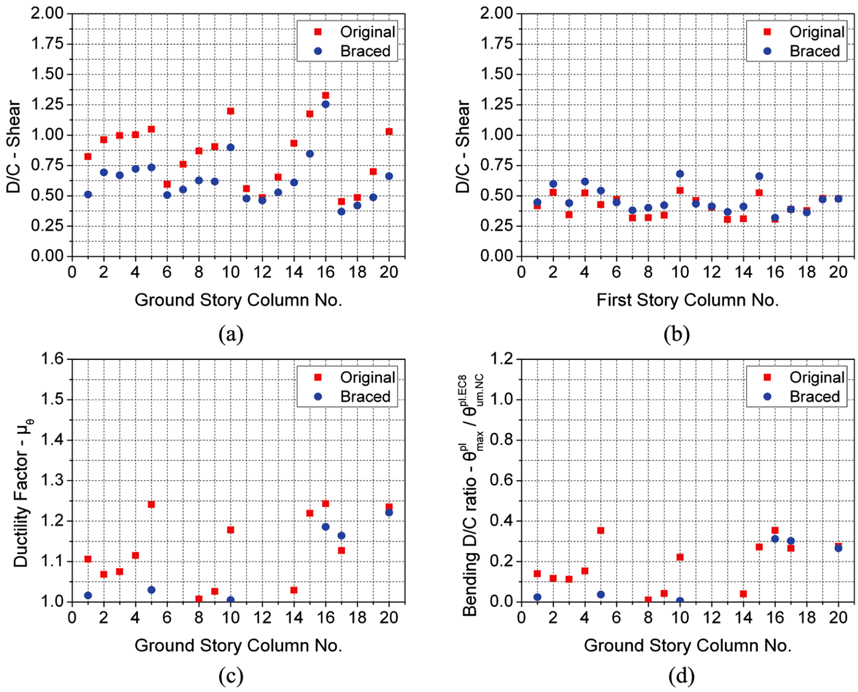

Fig. 12 5-story eccentric building: Shear *D*/*C* ratios in (a) ground story and (b) first story columns, (c) rotational ductility μ_θ and (d) bending *D*/*C* ratios in ground story columns

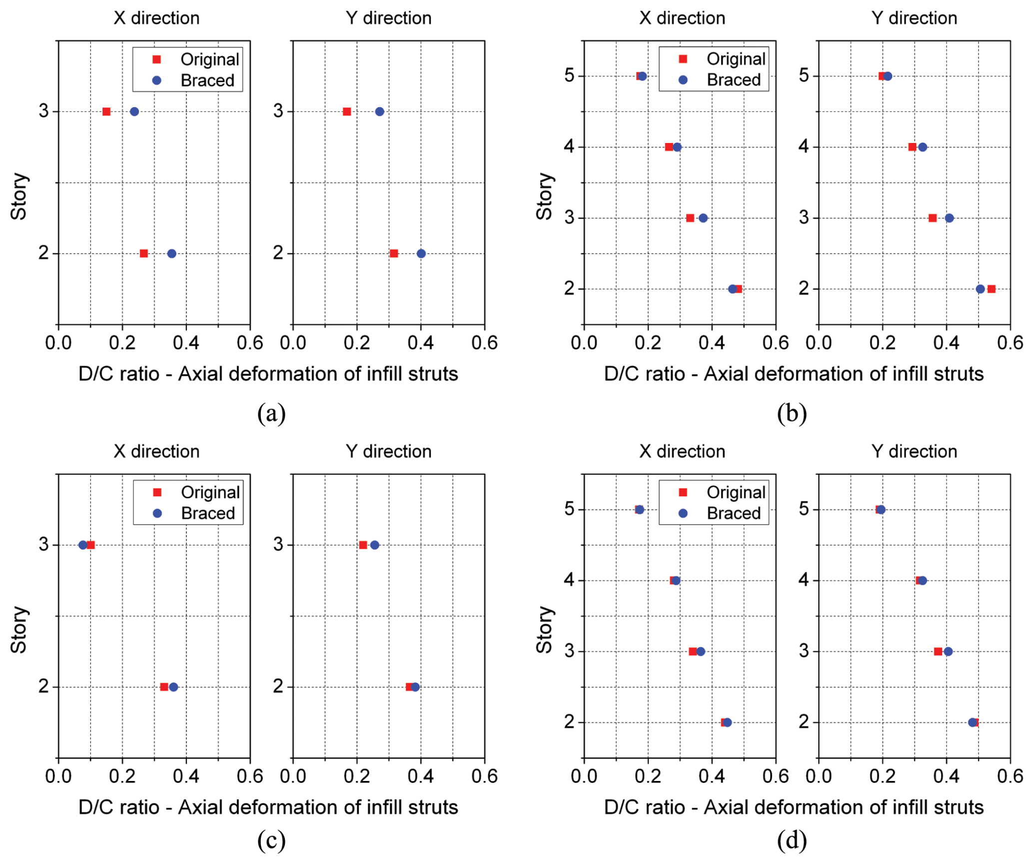

Fig. 13 *D*/*C* ratios in infills of the (a) 3-story symmetric, (b) 5-story symmetric, (c) 3-story eccentric and (d) 5-story eccentric building

Table 2 Maximum ductility factors of brace elements (tension only)

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

5. Conclusions

The work reported herein addresses the problem of strengthening the most vulnerable class of existing reinforced concrete 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. There are thousands of such buildings in Greek cities and their weak first story poses a significant risk for their performance in future earthquakes. 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, for two important reasons: (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. For the specific buildings examined, demand-to-capacity (D/C) ratios for shear and bending deformations of several ground story columns were greater than 1.0 in the original designs and were reduced to acceptable levels below 1.0 after strengthening, while the corresponding increases in the story above did not make any of these ratios greater than 1.0. 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 earthquake.

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APPENDIX – A

Table A1 3 story symmetric building: dimensions and reinforcement in ground-story columns

Ground floor		
Column No.	Dimensions ($b \times h$)	Reinforcement
1,5,16,20	30×30	4Φ20
2,4,6,10,11,15,17,19	35×35	4Φ18
7,9,12,14	40×40	4Φ20
8,13	45×45	4Φ18+4Φ16

Table A2 5 story symmetric building: dimensions and reinforcement in ground-story columns

Ground floor		
Column No.	Dimensions ($b \times h$)	Reinforcement
1,5,16,20	35×35	4Φ20
2,4,17,19	45×40	4Φ18+2Φ16
6,10,11,15	45×50	4Φ20+2Φ18
3,7,9,12,14,18	50×50	4Φ20+4Φ18
8,13	55×55	4Φ20+4Φ18

Table A3 3 story eccentric building: dimensions and reinforcement in ground-story columns

Ground floor		
Column No.	Dimensions ($b \times h$)	Reinforcement
1,5,16,20	35×35	4Φ20
2,4,18	35×30	4Φ18
3,7,9,12,14	40×40	4Φ20
6,10,11,15	35×40	6Φ16
8,13	45×40	6Φ18
17	200×150×20	16Φ14+2#8/20
19	30×30	4Φ16

Table A4 5 story eccentric building: dimensions and reinforcement in ground-story columns

Ground floor		
Column No.	Dimensions ($b \times h$)	Reinforcement
1,5,16,20	35×35	4Φ20
2,4,18	45×40	8Φ18
3,7,9,12,14	50×50	4Φ20+4Φ18
6,10,11,15	45×50	4Φ20+2Φ18
8,13	55×50	4Φ20+4Φ18
17	200×150×20	16Φ14+2#8/20
19	40×40	4Φ18+4Φ16

Table A5 Section profiles of steel bracing

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