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IMPROVING THE SEISMIC PERFORMANCE OF EXISTING OLD PILOTIS TYPE BUILDINGS BY STRENGTHENING ONLY THE GROUND STORY

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Abstract. *The present paper deals with the problem of partial strengthening of old Reinforced Concrete (RC) buildings with open ground floors (pilotis). The strengthening is restricted to the ground level, so that the cost is kept to a minimum and the building remains in use during the intervention. Since strengthening of the ground story will typically transfer the problem to the story or stories above, it is necessary to determine the amount of required strengthening so that the net result will be an optimum, i.e. the maximum possible reduction of the building's vulnerability. In essence the intervention is aimed at increasing the overall seismic resistance for the said class of buildings by removing the "soft story" weakness. This limited intervention is not generally expected to bring the building up to the standards of seismic safety implied by the new codes. For dealing with this problem, plane RC frames corresponding to symmetric buildings, as well as non-symmetric buildings in plan, representative of the old building stock in Greece, are designed in accordance with the old codes (RC code and Code for earthquake resistant design). Subsequently they are subjected to a set of code compatible accelerograms and using nonlinear time history analyses conclusions are drawn regarding the effectiveness of ground story strengthening. Evaluation of their seismic performance is based on Part-3 of Eurocode 8 for assessment and retrofitting of buildings. It is shown that strengthening only the open ground story can effectively reduce the vulnerability of pilotis type buildings by removing the inherent weakness. For non-symmetric buildings, appropriate selection of the bracing location and size, can also improve building performance by reducing the ground story stiffness eccentricity and the consequent torsional response.*

1 INTRODUCTION

Existing multistory reinforced concrete (RC) buildings with brick infills and open ground stories (pilotis buildings) designed by the Old Greek Codes applicable till 1984, represent a structural type that has suffered most of the heavy damage and collapses during strong earthquake events in Greece in the past 30 years (e.g. in the Alcyonides 1981, Kalamata 1986, Aigion 1995 and Athens 1999 earthquakes) and worldwide (e.g. in the Mexico 1985 and Kocaeli-Izmit 1999 earthquakes). Earthquake response of such buildings is strongly dependant on the behavior of their open ground stories. Modern Earthquake Resistant Design Codes for new structures include special provisions for buildings with vertical irregularities and a weak ground story is one of them. As an example, Eurocode 8 [1] 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.

The abrupt reduction in strength and stiffness due to the absence of infill walls in ground stories is a problem that unfortunately was not recognized by older Greek Codes. Combined with other code shortcomings and inadequate construction practices of the past, this major structural deficiency led to weaker than desired buildings, as numerically documented and witnessed by their performance in recent earthquakes [2, 3, 4]. 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 the total cost, while allowing the building to remain operational during the intervention.

Seismic evaluation and upgrading of open ground story buildings has been examined by many researchers and various retrofitting alternatives have been proposed. Fakhouri and Igarashi [5] proposed an assembly of multiple-slider surface bearings set in parallel on the top of the first story columns as an isolation interface for the seismic retrofit of existing buildings with inadequate soft first stories. Briman and Ribakov [6] examined a new engineering solution for seismic isolation of soft first story buildings, by replacing existing weak columns with special seismic isolation columns and proposed a method for the design of buildings incorporating these devices. Pinarbasi et al. [7] presented results of a parametric study on a simplified five story model, showing that conventional base isolation can improve the overall seismic performance of existing buildings with soft first stories, being also effective in reducing the seismic demands on the most vulnerable, ground story level. Finally, the application of energy dissipation braces on the ground story bays of an existing six story building examined by Mezzi [8], showed significant improvement of the entire seismic response of the building.

Among these alternatives, conventional steel X-diagonal bracing, remains a common technique for seismic strengthening, which provides considerable increase in strength and stiffness of the building. This paper presents results of a study on partial strengthening of RC buildings with steel braces restricted to the open ground story. The following building sets were selected and designed according to the old Greek codes: (a) four plane frames with 3 bays each and with 2, 3, 5 and 8 stories (b) two, 5-story plane frames, with 2 and 4 bays, and (c) two eccentric buildings with 3 and 5 stories. These buildings are subsequently strengthened by means of suitable X-bracing in selected bays of the ground story, and their seismic performance, before and after strengthening, is evaluated according to the provisions of Eurocode 8 Part-3 for assessment and retrofitting of buildings, using nonlinear time history analyses for a set of code compatible artificial accelerograms. 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 represent multistory RC structures on “pilotis”, having brick infill walls in all stories except the ground story. They are space frame structures with symmetric and non-symmetric (eccentric) in plan layouts. They were selected to represent typical Greek buildings on pilotis, constructed before 1984, year when the old codes were modified. Plane frames are used for the analyses of the symmetric buildings while three dimensional models are used to represent the non-symmetric ones. Figure 1 shows corresponding elevations of the six different plane frames considered in the study, indicating also the bays where steel braces are placed for strengthening. These frames correspond to orthogonal in plan buildings with geometrical (mass and stiffness) symmetry, in both principal directions. The masses and loads supported by each frame in the actual building are calculated considering a distance between adjacent (parallel) frames equal to 6m.

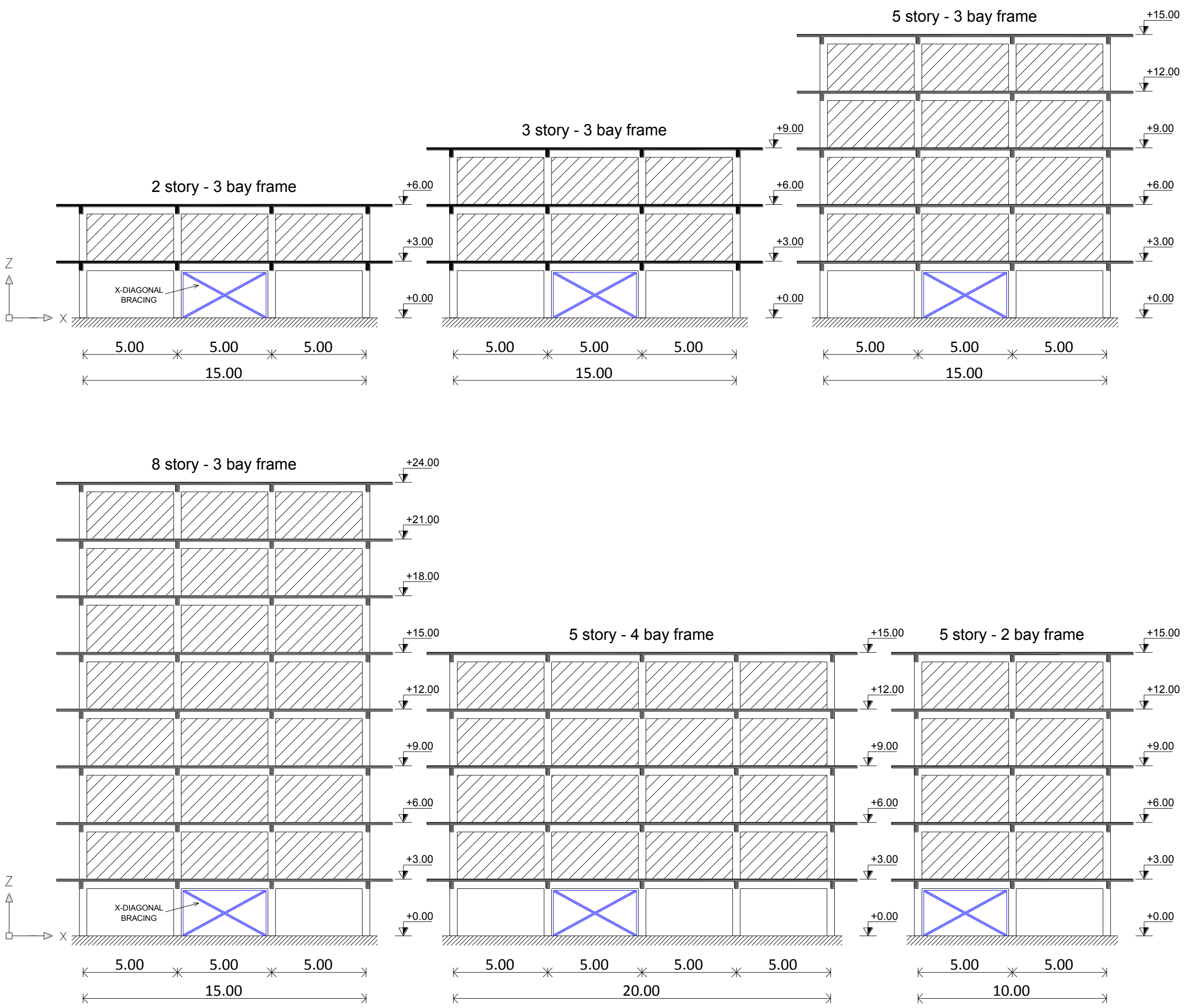

Figure 1: Typical elevations of the plane (2D) frames considered in the analyses.

Figure 2 shows the typical floor plan of both non-symmetric buildings (3 and 5 stories) and the elevations along x and y directions of the 5-story building. The elevator shaft, typical for similar old buildings, is located in the corner and causes 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 centre of mass and an approximate center of stiffness, estimated for all floors according to Stathopoulos and Anagnostopoulos [9], and normalized by the corresponding maximum building dimensions along the x and y axes. The selected bays of the open ground story where steel braces are placed for strengthening are also shown in the figures.

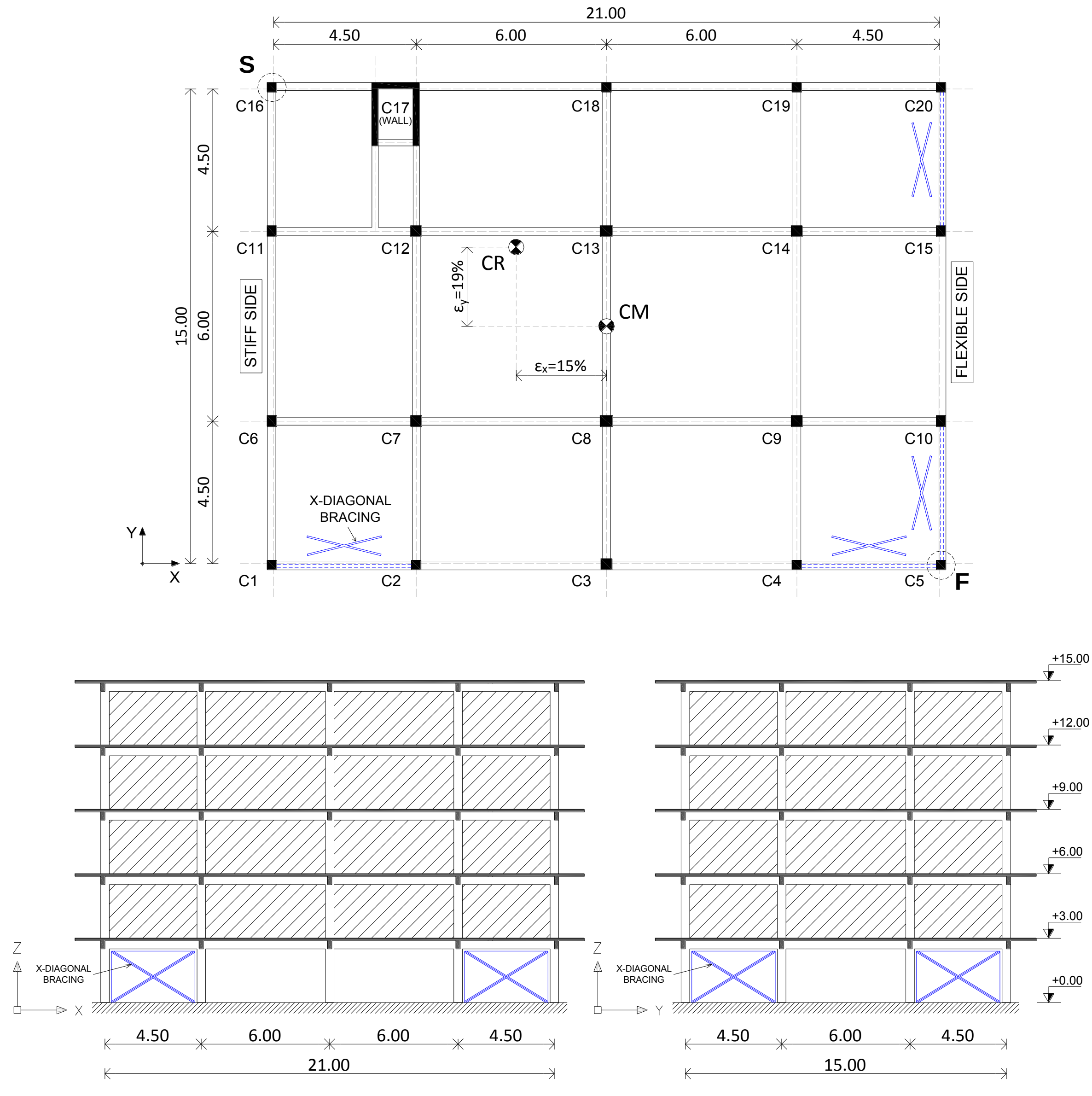

Figure 2: Typical layout of the non-symmetric 3 and 5 story buildings (top) and corresponding elevations along x and y directions of the 5-story building (bottom).

The original buildings have been designed in accordance with the old Greek Codes for reinforced concrete and for earthquake resistant design. A base shear coefficient equal to ε=0.04 was selected for seismic actions, corresponding to the lowest seismic zone (i.e. Zone I) and Soil Class A (rock) of the old 1959 Code, and thus, the design base shear of each building is equal to 4% of the total G+P (permanent plus live) gravity loads. Member dimensioning and corresponding design checks were done according to the allowable stress method of the old 1954 Reinforced Concrete Design Code 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. Simply supported continuous beams were assumed for the gravity loads, while the seismic forces of the columns and the shear walls were determined story by story, assuming no joint rotations (i.e. shear beam model).

Longitudinal steel reinforcement ratios for columns ranged between 0.8% and 1.1% of the gross section area, while transverse reinforcement consisted of smooth steel stirrups, 6mm in diameter, with open, 90° hooks, equally spaced at 20cm along the entire column length (non-seismically detailed transverse reinforcement). Longitudinal reinforcement of beams in all buildings was controlled mainly by gravity loads. For shear reinforcement in beams, 8 mm stirrups equally spaced at 15 and 25cm was provided in the beams of the frames and the eccentric buildings, respectively, according to the design.

Table 1 lists the section profiles of the steel braces used for the seismic strengthening of the buildings in the selected bays of their open ground stories.

Building	X – direction	Y – direction
2 story – 3 bay frame	CHS-133-4*	–
3 story – 3 bay frame	CHS-114.3-4.5	–
5 story – 3 bay frame	CHS-101.6-4.5	–
8 story – 3 bay frame	CHS-88.9-4	–
5 story – 2 bay frame	CHS-88.9-3.2	–
5 story – 4 bay frame	CHS-139.7-4	–
3 story eccentric building	CHS-114.3-5.6	CHS-114.3-3.6
5 story eccentric building	CHS-88.9-5	CHS-88.9-4

* Circular Hollow Section: 133mm diameter, 4mm thickness

Table 1: Section profiles of the steel braces of the buildings.

The brace sections listed above were selected after a preliminary analysis with the objective not to overdesign (over-strengthen) 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 buildings for the selected earthquake actions. As illustrated in the subsequent sections, this goal was met and the buildings' performance was significantly improved.

3 NONLINEAR MODELING AND EARTHQUAKE INPUT

Seismic behavior of the buildings before and after strengthening was evaluated using nonlinear time history analyses, based on Part-3 of Eurocode 8 [10] for assessment and retrofitting of buildings. Seven pairs of artificial accelerograms were generated using the code by Halldorsson et al. [11]. The selected motions comply with the rules of EC8 [1] for time history representation of the seismic action, i.e. their 5% damped average response spectrum matches the target design spectrum of EAK [12] for seismic Zone I (PGA=0.16g) and Soil Class A (Rock), as illustrated in Figure 3.

The nonlinear dynamic analyses of the buildings were carried out using the computer program Ruaumoko 3D [13]. Beams, columns and the elevator shaft (wall) were modeled using prismatic frame elements, while brick infill walls and the steel braces were modeled using special spring elements. Effective stiffness of RC members was taken equal to the secant stiffness at yield, based on mean material strengths (f_{cm}=12.8Mpa for B160 concrete, f_{ym}=253Mpa for St I grade longitudinal steel reinforcement and stirrups), according to EC8-Part 3 [10]. 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 analyses. 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 and Carr [14]. Based on data by Karantoni [15] for masonry bricks, the mean value of the compressive strength of the infills in the direction of the diagonal, was calculated equal to f_{wm}=2.3Mpa according to the provisions of the Greek Retrofitting Code [16] for masonry infills, and the corresponding strain was chosen equal to ε_w=0.0015. A constant width equal to 15% of the clear diagonal length was chosen for the infill struts with a thickness equal to 0.20m. The calculated strength of the infills, and thus the corresponding stiffness, was reduced to its half value for each panel of the plane frames, in order to account for any possible openings as well as for the influence on the global response of plane frames with no infills in the upper stories, parallel to those examined. Regarding the eccentric buildings, an approximate reduction equal to 60% of the initial strength of the infills was applied only in the panels where openings were considered, i.e. at selected bays in the perimeter of the buildings. For linear modal analysis, both struts are active, each assumed to work with half its stiffness. For nonlinear analysis, infill wall struts work only in compression and their axial stiffness is controlled by the hysteresis rule.

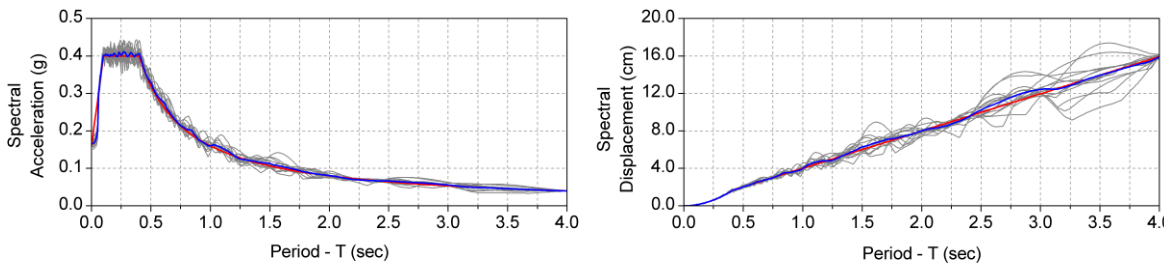

Figure 3: Mean response versus target design pseudo-acceleration (left) and displacement (right) spectra for the 14 synthetic accelerograms.

Diagonal steel cross-bracing members were also modeled with spring elements, following bilinear force deformation relationships. Brace tensile strength was calculated according to Eurocodes 3 and 8 [17, 1], while compressive strength was taken as a fraction of the buckling load (20%), according to the Greek Retrofitting Code [16]. Masses for the dynamic degrees of freedom were calculated from the quasi permanent static combination (G+0.3Q) and considered lumped at 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.

Fundamental periods of vibration and effective modal masses of the plane frames before and after strengthening are listed in Table 2. Looking at the effective modal masses of the original buildings, especially of those with fewer stories, we see that the presence of “pilotis” almost transforms the building to a single degree of freedom system, with nearly the total mass of the system vibrating in the fundamental mode.

Frame	Period T (sec)		Modal mass M^*_x (%)	
	original	braced	original	braced
2 story – 3 bay	0.609	0.268	99.6	87.0
3 story – 3 bay	0.653	0.366	98.6	84.4
5 story – 3 bay	0.772	0.604	93.5	82.4
8 story – 3 bay	1.053	0.965	86.4	80.8
5 story – 2 bay	0.770	0.615	92.9	82.4
5 story – 4 bay	0.766	0.606	93.4	82.9

Table 2: Modal data for the fundamental modes of the original and the braced frames.

Table 3 lists the modal data for the first three modes of the original and the braced eccentric buildings, along x and y directions. The potential for torsional response of these buildings is reflected in the effective modal mass ratios. The addition of steel braces at sides opposite to the stiff elevator shaft (see, Fig. 2 (top)) reduces the eccentricity and the resulting torsion, as can be inferred by comparing the effective modal mass ratios of the buildings before and after strengthening (original vs. braced).

Building	Mode	Period T (sec)		Modal mass M^*_x (%)		Modal mass M^*_y (%)	
		original	braced	original	braced	original	braced
3st – eccentric	1	0.664	0.461	33.0	8.0	31.0	76.0
	2	0.517	0.420	44.0	78.0	49.0	9.0
	3	0.418	0.336	17.0	1.0	13.0	2.0
5st – eccentric	1	0.829	0.709	32.0	2.0	38.0	80.0
	2	0.728	0.673	39.0	79.0	44.0	2.0
	3	0.592	0.542	16.0	2.0	4.0	0.0

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

4 NONLINEAR ANALYSES RESULTS

Seismic performance of the buildings before and after strengthening (original vs. braced) was evaluated according to EC8-Part 3 [10]. Member verifications were carried out for all components (beams, columns and walls), both for flexure and shear. The design seismic action for which buildings were analyzed corresponds to the Limit State of Significant Damage. Each building was analyzed for the selected ground motions and peak response quantities were calculated through step by step post processing of large sets of time history analysis results. One accelerogram of each motion pair was used for the nonlinear dynamic analyses of each 2D frame, resulting in a total of 7 sets of results, while in the case of the 3D eccentric buildings, each motion pair was used twice by mutually interchanging the accelerograms in the x and y principal directions, resulting in a total number of 14 analyses and 14 sets of results. The results presented in the following pages are the mean values of the maxima from 7 (plane frames) or 14 (eccentric buildings) sets of analyses.

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

- Maximum displacements along building height (in absolute values).
- Interstory drifts (relative story displacements, in absolute values).

- Rotational 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 EC8-Part 3 [10]. Yield rotations θ_y for each member were assumed constant, and equal to those initially calculated under the action of the quasi permanent gravity loads for the determination of the effective stiffnesses of the members to be used in the mathematical models.

- Demand to capacity ratios (D/C) of the maximum hinge rotations to the instantaneous (due to variation of the axial loads) plastic rotation capacity of the members with smooth longitudinal bars, without detailing for earthquake resistance, based on mean material strengths, calculated according to EC8-Part 3 [10]. Strength and deformability modification due to lap-splicing of the column reinforcements 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 axis 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

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

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

- 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 EC8-Part 3 [10]. The contribution of stirrups to the calculations of cyclic shear resistance was reduced to half its calculated value, due to open (non-seismically detailed) stirrups, see Biskinis et al. [18].

Peak story displacements and maximum interstory drifts of the original and the braced plane frame buildings are shown in Figures 4 to 9. Looking at the response of the original buildings (red dashed lines), a clear soft story behavior is apparent, because the largest portion of the total displacements is concentrated at the open ground story. In the 8-story frame the soft ground story drift is not as dominant as in the other frames, nevertheless it still remains the largest over all stories (Fig. 7).

Comparing the response of the buildings before and after strengthening, we observe a significant reduction of the ground story displacements, with relatively limited increase in the displacements of the stories above. It is worth to note that in some cases, in addition to significantly reducing the ground story displacements, the steel bracing reduces also the displacements of the first floor above, which may be attributed, to some extent, to the energy dissipated after braces yield in tension.

The results of the member checks are even better. This can be seen in Table 4 summarizing basic quantities for the overall seismic safety of the original and the braced buildings, i.e. maximum values of the shear D/C ratios of the ground and first story columns, maximum ductility demands, bending D/C ratios of the plastic hinge rotations of the ground story columns and D/C ratios of infill walls just above the ground story.

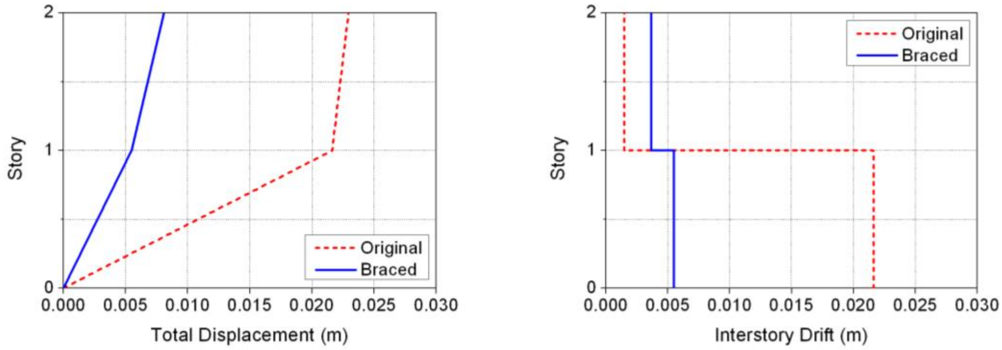

Figure 4: 2story-3bay frame, average values of maximum total displacements and maximum interstory drifts.

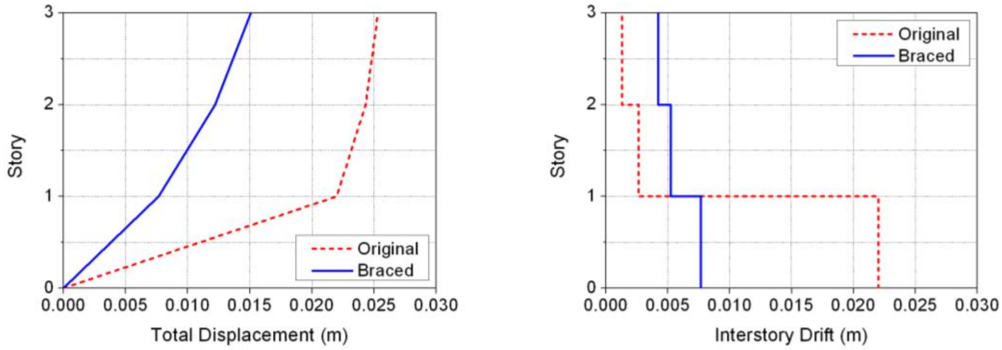

Figure 5: 3story-3bay frame, average values of maximum total displacements and maximum interstory drifts.

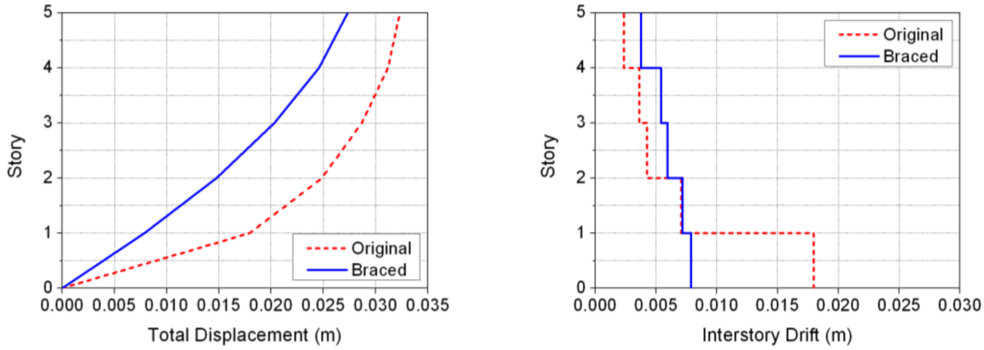

Figure 6: 5story-3bay frame, average values of maximum total displacements and maximum interstory drifts.

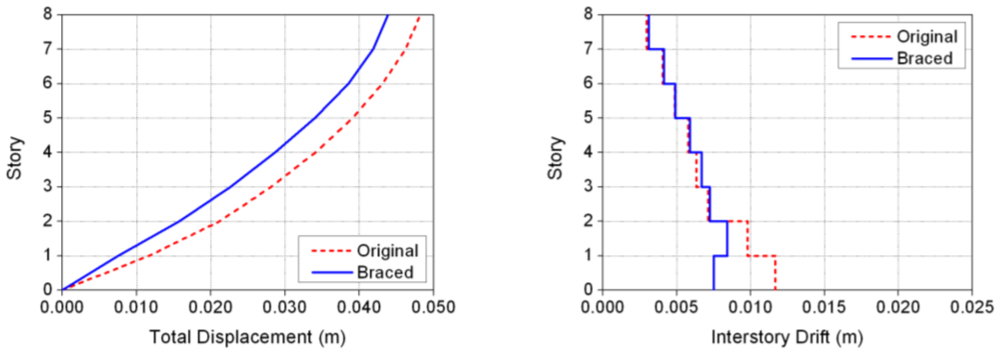

Figure 7: 8story-3bay frame, average values of maximum total displacements and maximum interstory drifts.

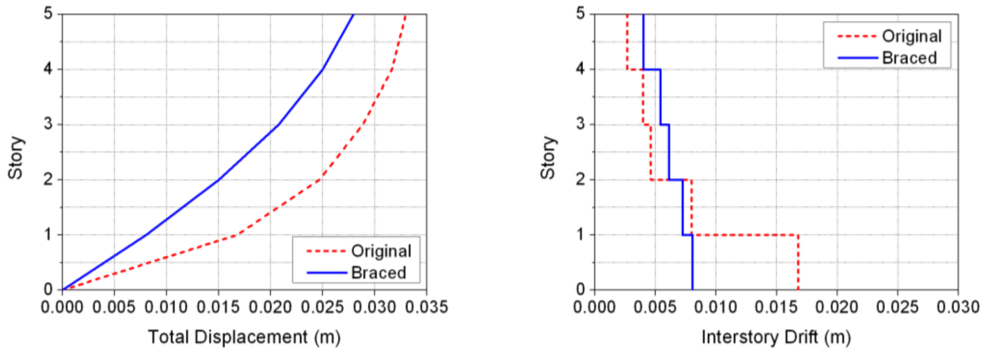

Figure 8: 5story-2bay frame, average values of maximum total displacements and maximum interstory drifts.

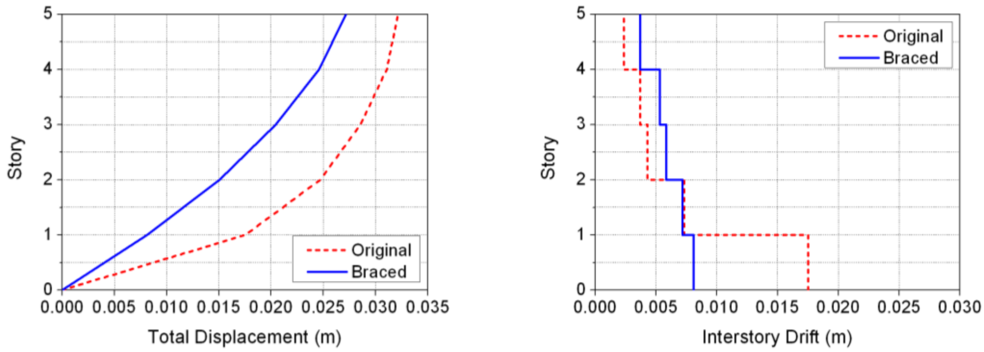

Figure 9: 5story-4bay frame, average values of maximum total displacements and maximum interstory drifts.

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 buildings, indicative of high risk of failure, now were reduced to values below 1.0, as may be seen in Table 4. The expected increase of these ratios in the columns of the story just above the ground story is also shown in Table 4, but their values are still bellow 1.0.

Frame		D/C - V_R Pilotis Columns	D/C - V_R 1st Floor Columns	μ_θ Pilotis Columns	D/C - θ_{pl} Pilotis Columns	D/C ratio 1st Floor Infills
2 story – 3 bay	original	1.250	0.818	1.286	0.370	0.170
	braced	0.621	0.847	-*	-	0.406
3 story – 3 bay	original	1.272	0.628	1.343	0.459	0.301
	braced	0.689	0.716	-	-	0.579
5 story – 3 bay	original	1.334	0.570	1.242	0.410	0.737
	braced	0.734	0.697	-	-	0.783
8 story – 3 bay	original	0.995	0.632	1.010	0.012	1.078
	braced	0.715	0.676	-	-	0.944
5 story – 2 bay	original	1.328	0.630	1.192	0.238	0.806
	braced	0.806	0.707	-	-	0.793
5 story – 4 bay	original	1.323	0.579	1.198	0.360	0.762
	braced	0.743	0.691	-	-	0.787

*columns remained elastic during the response (no hinges were formed)

Table 4: Maximum member D/C ratios of the original and the strengthened frame buildings.

Regarding the masonry infills, which inevitably play their role on the global seismic response, their damage index (infills D/C ratio) was taken equal to the ratio of the maximum axial deformation to the deformation at maximum strength of each infill. Conventionally, values greater than 1.0 correspond to infills that have reached their maximum available strength and have started to degrade. After calculating this index for the two infills (springs) of each panel separately, average values among all infills in the story were calculated as global story infill damage indices.

Displacement results for the 3 and 5 story eccentric buildings are shown in Figures 10 and 11, respectively. They are given for the "stiff" and the "flexible" edges of the buildings at points "S" and "F", respectively (see, Fig. 2 (top)). Overall, the behavior 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, where the displacements due to torsion are added to the translational displacements due to the earthquake lateral force. This is clearly seen in the response of the original 3-story eccentric building, while it is less evident in the case of the 5-story one. Contrary to the symmetric in plan buildings, where steel braces can be placed in any of the available bays provided that the symmetry in plan is maintained, 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, a torsional response is still apparent.

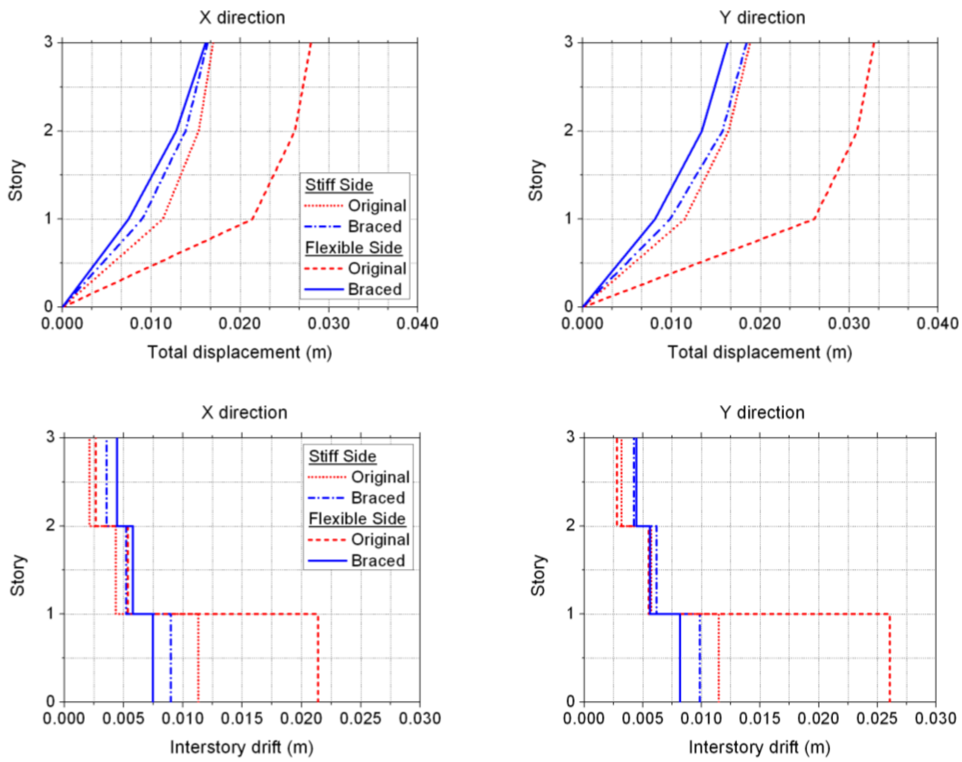

Figure 10: 3-story eccentric building, average values of maximum total displacements (top) and maximum interstory drifts (bottom) along X and Y direction.

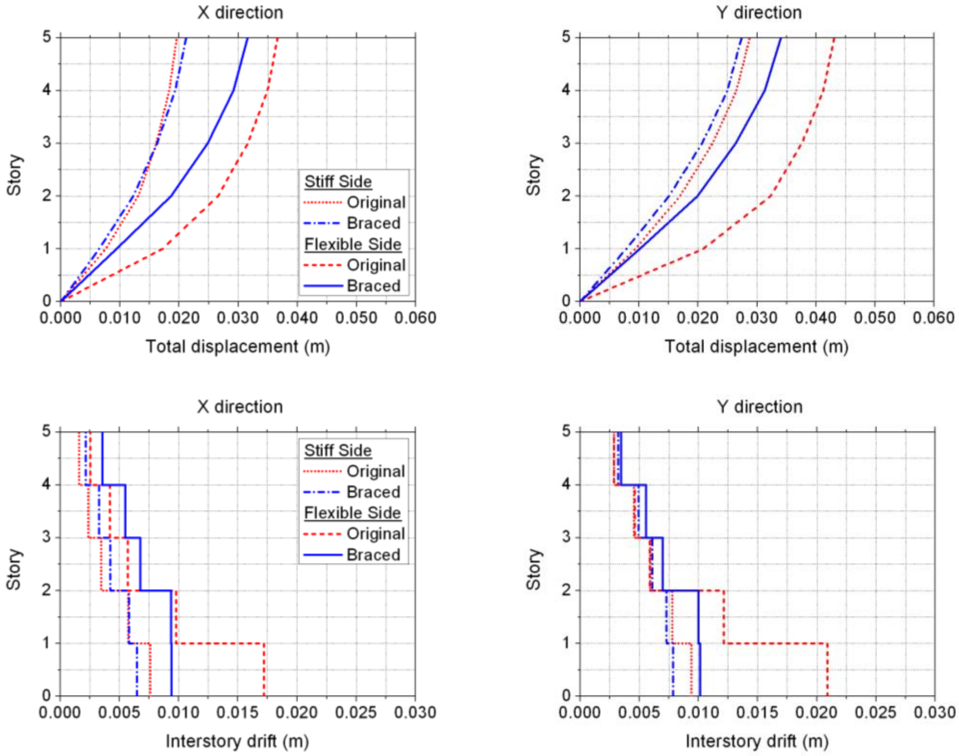

Figure 11: 5-story eccentric building, average values of maximum total displacements (top) and maximum interstory drifts (bottom) along X and Y direction.

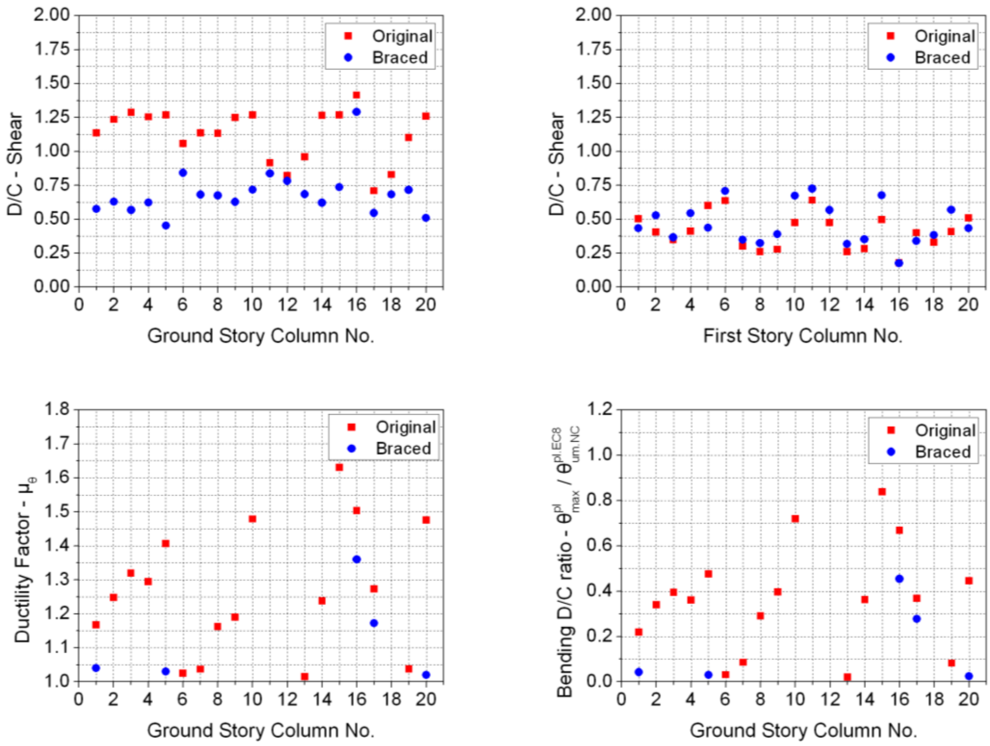

Figure 12: 3-story eccentric building, shear D/C ratios in ground story and first story columns (top), rotational ductility μ_θ and bending D/C ratios in ground story columns (bottom).

In order to have a better picture of the seismic performance of the eccentric buildings before and after strengthening, member checks are presented in a more detailed fashion, as illustrated in Figures 12 and 13. In addition D/C ratios for infill walls in all stories above the pilotis are shown in Figure 14. Looking at Figures 12 and 13, it is clear that steel bracing reduces significantly the potential for shear failures in the ground story columns. We note that even after the intervention, one column in the ground story of the eccentric buildings fails in shear and a few others suffer damage as the rotational ductility factors indicate (see, Fig. 12 and 13). Such cases should be dealt with local measures, e.g. using concrete jackets, because further strengthening of the ground story will create problems in the stories above.

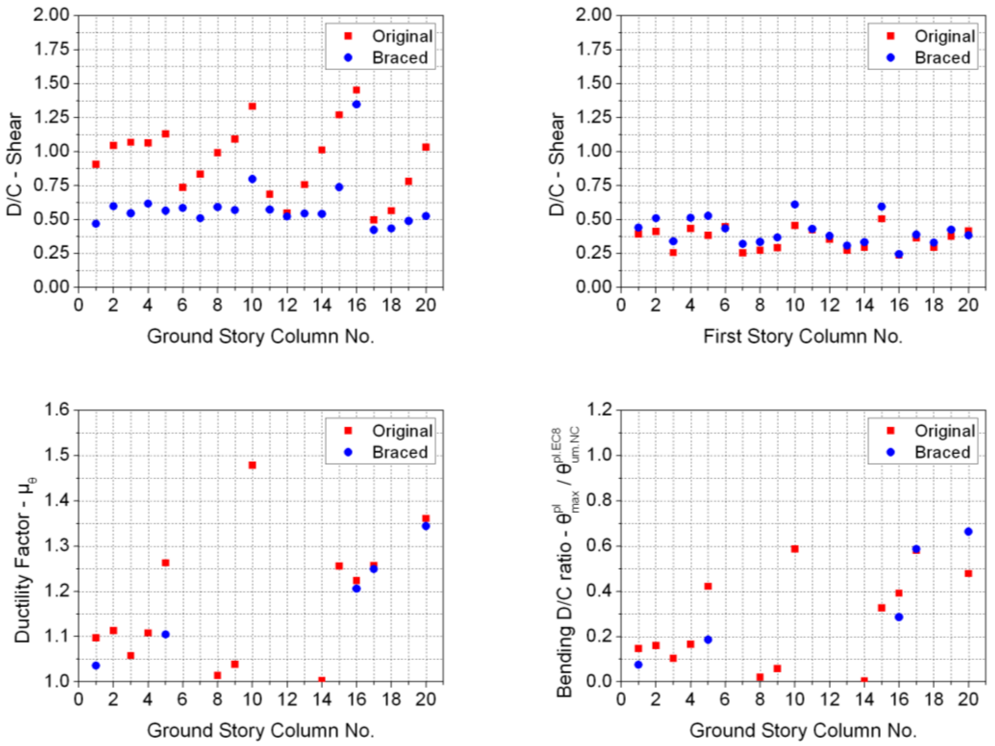

Figure 13: 5-story eccentric building, shear D/C ratios in ground story and first story columns (top), rotational ductility μ_θ and bending D/C ratios in ground story columns (bottom).

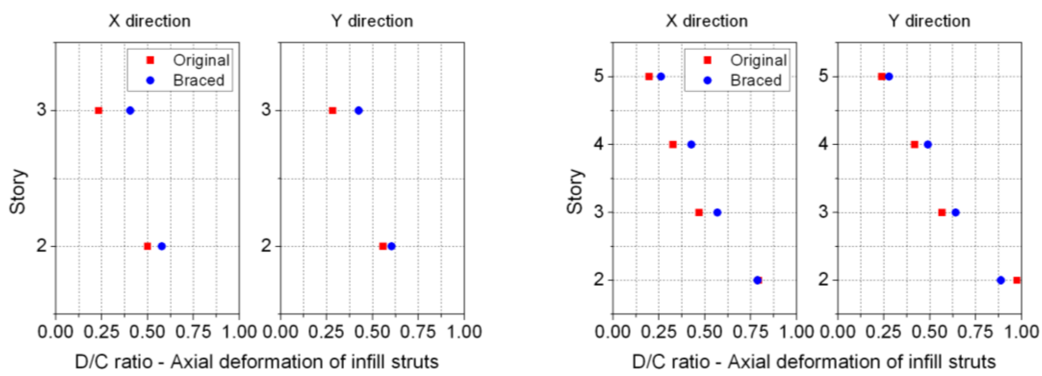

Figure 14: D/C ratios in infills of the 3-story (left) and 5-story eccentric building (right) along X and Y directions for all floors above the ground story.

For the evaluation of the steel braces maximum ductility factors in tension were computed for each brace and averaged over the number of analyses performed. The maximum over all the braces is the value listed in Table 5. As may be seen in the Table, the values of these ductility factors are relatively low, compared to the values expected in braced steel frames (in the case of the 2-story frame the braces remained elastic during the response).

Building	X – direction	Y – direction
2 story – 3 bay frame	0.783	–
3 story – 3 bay frame	1.119	–
5 story – 3 bay frame	1.127	–
8 story – 3 bay frame	1.106	–
5 story – 2 bay frame	1.150	–
5 story – 4 bay frame	1.160	–
3 story eccentric building	1.083	1.179
5 story eccentric building	1.377	1.506

Table 5: Average values of the maximum ductility factors of brace elements (tension only).

It is worth pointing out that the use of braces for seismic strengthening of RC structures is effective for global strengthening, provided that a reliable, well detailed and technically sound connection between the steel elements and the existing concrete buildings is ensured. Dimensioning of all the extra members (steel rims, anchors, headed studs, mortar joints e.t.c) required for the attachment of the steel bracing to the perimeter of each RC frame, is beyond the scope of this work.

5 CONCLUSIONS

The work reported herein looks into 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 strong earthquakes of the recent past. The present paper examined the feasibility of partial strengthening of such buildings, aiming at reducing their vulnerability due to 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. Based on inelastic dynamic earthquake response analyses of a set of plane frames, used to form symmetric buildings in plan, as well as two eccentric buildings, their vulnerability due to weak ground story was first confirmed. These buildings were subsequently strengthened with steel braces placed in appropriately selected bays of the ground story, and their performance was examined under the same earthquake action. In all cases the strengthened buildings showed significantly improved earthquake response, which met the initial objective of removing the ground story weakness without shifting the problem to higher stories. Note also that with the selected bracing locations in the case of non-symmetric buildings, it was possible to reduce the ground story eccentricity, and through that, the undesirable torsional-soft story response of the buildings. It is believed that partial strengthening by intervening only in the open ground story, as opposed to complete strengthening to comply with current standards for new buildings, is perhaps the only feasible way of intervention in the existing stock of the most vulnerable old buildings: first due to its low cost (that becomes even lower and hence affordable if split among the building several owners) and second because the building can remain functional during the intervention works.

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