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Seismic Retrofitting Strategy for Improved Strength and Ductility of a Plan-Wise Irregular Reinforced Concrete Building

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Abstract

A mixed retrofitting intervention including both fibre reinforced polymer (FRP) wrapping and reinforced concrete (RC) jacketing applied to selected columns was proposed and investigated by numerical analyses with the aim of improving the seismic performance of a four-storey plan-wise irregular RC building designed for gravity loads. Retrofitting was aimed at both reducing the torsional component of the seismic response and improving the local and global ductility of the structure. A displacement-based procedure using nonlinear static pushover analyses was adopted to assess the seismic performance of the structure in the original configuration and to select the retrofitting intervention. As a result of the asymmetry of the investigated structure, appropriate correction factors were computed in order to take into account the effects of torsion. Nonlinear dynamic analyses were carried out to verify the effectiveness of the retrofitting intervention strategy. Demand-to-capacity ratio (DCR) values were used to evaluate the damage level of columns and to identify the most critical columns affecting the seismic performance of the structure.

Keywords: seismic retrofitting, reinforced concrete building, irregularity, nonlinear static analyses, nonlinear dynamic analyses.

1 Introduction

A seismic retrofitting strategy for improved strength and ductility of an under-designed plan-wise irregular reinforced concrete (RC) building was investigated by nonlinear dynamic analyses and by simplified procedures based on nonlinear static pushover analyses. Appropriate correction factors were computed in order to take into account the effects of torsion due to the asymmetry of the investigated structure. The retrofitting intervention strategy was based on the decrease of the torsional component highlighted in the seismic response of the original structure by means of the reduction of the eccentricity of the centre of stiffness (CR) with respect to the centre of mass (CM). The strength and stiffness relocation was achieved using the traditional technique of RC jacketing, limited to selected columns. The mixed retrofitting intervention included FRP wrapping applied to the other columns with the aim of improving the local and global ductility of the structure. The results of the numerical investigations showed that the combination of the two approaches (RC jacketing and FRP wrapping) applied to selected columns significantly improved the seismic performance of the structure, increasing strength, stiffness and ductility.

2 Building under study and numerical modelling

The procedure for seismic assessment and retrofitting of structures was developed within a displacement-based approach and was applied to a four-storey RC building, designed for gravity loads without the application of specific earthquake-resistant provisions. Figure 1 shows the plan and the elevation of the RC building under study. The materials used in the design were concrete C20/25 and steel S400 for longitudinal and transversal reinforcement. Dead loads consisted of the weights of structural components, infill walls and slab overlays, and live loads were considered to be equal to 2 kN/m^2. Storey masses included dead loads and a percentage of live loads (30% according to Eurocode 8 for common residential and office buildings). The columns presented square cross-sections of dimensions 30cm x 30cm, except the large column C2 with a rectangular cross-section of dimensions 30cm x 80cm. The large column C2 provided the structure with more stiffness and strength in the x direction than in the y direction. The beam cross-section dimensions were 30cm x 50cm. The eccentricities between the centre of mass (CM) and the centre of stiffness (CR) amounted to 0.22 m and 3.92 m (about 1.5% and 26% of the plan dimensions) in the x and y directions, respectively.

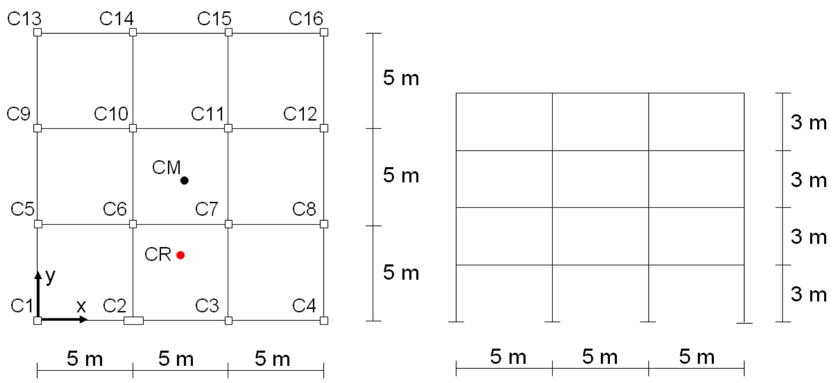

Figure 1. Schematic plan and elevation of the RC building and position of the centre of mass (CM) and the centre of stiffness (CR).

The RC building was modelled by using the computer code SeismoStruct [8]. The spread of the inelastic behaviour along the length of any member and within its cross-section was described by means of a fibre model, that made it possible to accurately evaluate the damage distribution. The sectional stress-strain state of inelastic frame elements was obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres into which the section was subdivided. Each member (column or beam) was subdivided into a number of elements, whose length was critical to effectively capture the expected inelastic behaviour of the dissipative zones of the structure. Concrete was modelled by using a uniaxial constant-confinement model based on the constitutive relationship proposed by Mander et al. [9], and later modified by Martinez-Rueda and Elnashai [10], to cope with some problems concerning numerical stability under large displacements. The confinement effects, provided by the transverse reinforcement, were taken into account through the rules proposed by Mander [9], whereby a constant confining pressure was assumed in the entire stress-strain range. The model required the introduction of four parameters: the compressive and tensile strengths of the unconfined concrete, the crushing strain and the confinement factor (defined as the ratio between the confined and unconfined compressive strength of the concrete). In the case investigated here, the amount of transverse reinforcement of all members was very small to produce any effective confinement on the concrete. Due to the insufficiency of stirrups, the confinement factor was assumed to be close to 1 for all members in the numerical model. The constitutive model used for the longitudinal reinforcement was that proposed by Menegotto and Pinto [11], coupled with the isotropic hardening rules proposed by Filippou et al. [12]. Figure 2 shows a three-dimensional view of the numerical model of the RC building.

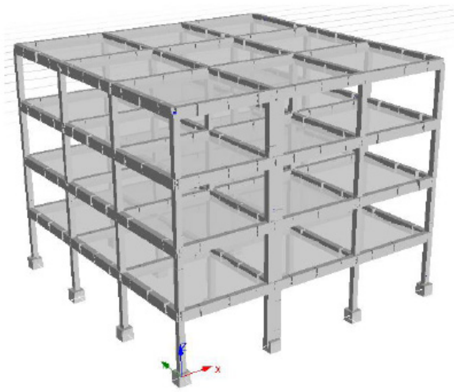

Figure 2. Three-dimensional view of the numerical model of the RC building.

3 Seismic performance assessment of the building

A simplified assessment procedure [6] was adopted for the seismic verification of the global structural behaviour of the RC building. This simplified method is an effective technique for the seismic assessment of existing structures and combines pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) model. The method is formulated in the acceleration-displacement (AD) format, which enables the visual interpretation of the results. By means of a graphical procedure, the seismic demand is compared with the capacity of a structure for different limit states. According to the requirements of Eurocode 8 Part 3 [3], the level of damage in the structure was evaluated with reference to three Limit States

(LS): Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). In the structural model, each limit state is achieved once a specific chord rotation is attained in one of the members of the structure: the LSDL, the LSSD and the LSNC correspond to the first attainment of θ_y, $0.75 \cdot \theta_u$ and θ_u, respectively. The expressions of the yielding (θ_y) and ultimate (θ_u) chord rotations are reported in Eurocode 8 Part 3 [3]. According to the code, in this study the most critical member was conservatively assumed to control the behaviour of the structure.
Nonlinear static pushover analyses were performed using the computer code independently in the two horizontal directions and a load in the positive and negative direction was taken into account. The distribution of the horizontal forces along the height of the building was determined using the fundamental mode which was critical for the particular direction. The bilinear idealization of the pushover curve with zero post-yield stiffness was defined on the basis of the "equal-energy" concept (the areas underneath the actual and idealized bilinear curves are approximately the same, within the range of interest).
The seismic demand was evaluated with reference to Eurocode 8 response spectrum (Type 1, subsoil class C) with $a_g = 0.25$g. The seismic assessment of the structure was performed by comparing seismic demand and capacity. The target displacement at the LSSD was computed as the intersection between the bilinear capacity curve and the inelastic demand spectrum characterized by the relevant ductility. The inelastic displacement demand was equal to the elastic displacement demand according to the equal-displacement rule, because the period of the equivalent SDOF system was larger than the characteristic period $T_C = 0.6$ s.
Due to the asymmetry of the investigated structure, appropriate correction factors were used in order to take into account the effect of torsion for plan-asymmetric building structures, as proposed by Fajfar [7]. The results obtained by pushover analysis were combined with the results of a linear dynamic (spectral) analysis. The target displacements and the distribution of deformations along the height of the building were determined by means of the simplified procedure, which is based on pushover analysis, whereas the torsional amplifications were determined by linear dynamic analysis in terms of correction factors to be applied to the relevant results of pushover analyses. The correction factor was defined as the ratio between the normalized roof displacements (the roof displacement d at an arbitrary location divided by the roof displacement d_{CM} at CM) obtained by linear dynamic analysis and by pushover analysis. Displacement reductions due to torsion were neglected. Torsional amplifications were taken into account for the columns of the flexible sides of the structure. Figure 3 presents the normalized roof displacements of the structure for linear dynamic analyses and pushover analyses at LSSD in the x and y directions.
Figure 4 shows that the bare structure was unable to satisfy the demand in both directions at a peak ground acceleration of $Sa_g = 0.29$g (S = soil factor) at the Limit State of Significant Damage. The displacement demand and capacity in Figure 4 refer to the equivalent SDOF system. The displacement demand and capacity of the MDOF system were obtained by multiplying the SDOF system demand and capacity by the transformation factor Γ. The difference between the seismic demand and the displacement capacity was 3.6 cm (15.1 cm *vs* 11.5 cm) in the x direction and 3.7

cm (16.5 cm *vs* 12.8 cm) in the y direction. The comparison of the bilinear idealized capacity curves of the structure in the x and y directions shows an increase of strength and stiffness in the x direction due to the orientation of the rectangular column C2. The simplified assessment procedure established that the critical columns were the internal columns C6, C7, C10, C11 with high axial load and the perimeter columns C14, C15, C16, C12 of the flexible edges with high torsional amplifications.

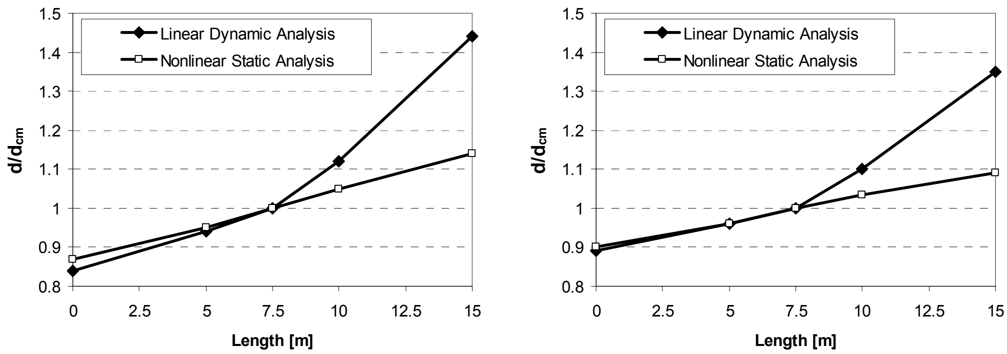

Figure 3. Normalized displacements at the top of the bare structure for linear dynamic analyses and nonlinear static analyses: x direction (left) and y direction (right).

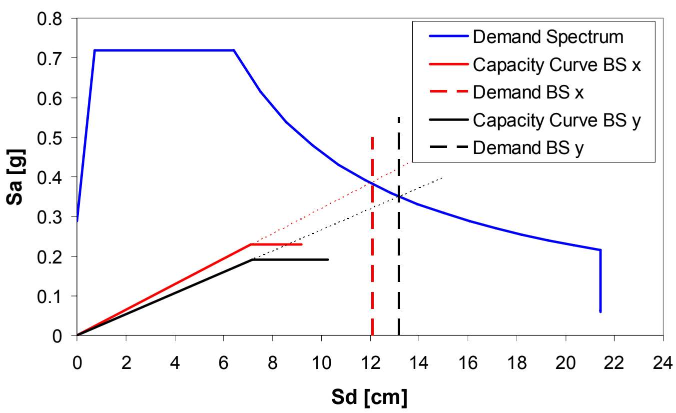

Figure 4. Demand spectrum and capacity curves in AD format at LSSD (Sa_g = 0.29g) for the bare structure in the x and y directions.

4 Design strategy for retrofitting intervention

A retrofitting intervention using both RC jacketing and glass-fibre-reinforced polymer (GFRP) laminates was carried out in order to improve the seismic performance of the structure. Figure 5 presents a schematic view of the proposed retrofitted structure, hereafter named as "RS1". The perimeter columns C5, C9, C14, C15, C12 and C8 were strengthened at all storeys with 20 cm-thick jackets,

longitudinally reinforced with 12∅16 bars. The ductility of these columns was increased by adding ∅10 stirrups, spaced by 100 mm. At all storeys, the remaining square columns were confined at the top and at the bottom by means of GFRP uniaxial laminates (thickness = 0.7 mm; modulus of elasticity = 72 GPa; tensile strength = 2000 MPa; ultimate strain = 0.035) in order to enhance structural ductility. The ultimate chord rotation of the retrofitted columns increased by about 70% with respect to the original columns. Quadriaxial GFRP laminates were used for the rectangular column C2, wrapped for the entire height at all storeys, in order to increase its shear capacity.
The combination of the two approaches (RC jacketing and FRP wrapping) applied to selected columns aimed at improving the seismic performance of the structure. The selection of the retrofitting intervention was based on the deficiencies underlined by numerical analyses performed on the bare structure. The retrofit strategy was focused on two main objectives: 1) relocating the centre of stiffness (CR) in order to reduce the torsional component of the response and increasing the strength and stiffness of the structure; 2) increasing the local deformation capacity of columns and thus the global deformation capacity of the structure. In the retrofitted structure the eccentricity of CR with respect to CM was significantly reduced compared to the bare structure and amounted to 0.06 m and 0.51 m in the x and y directions, respectively. Such a retrofitting intervention turned out to be very effective, since a sizable reduction of the torsional response was achieved in a rather simple way. The results of the free vibration analysis showed that the first two modes were predominantly translational (the first mainly in the y direction and the second mainly in the x direction), whereas the third mode was torsional.

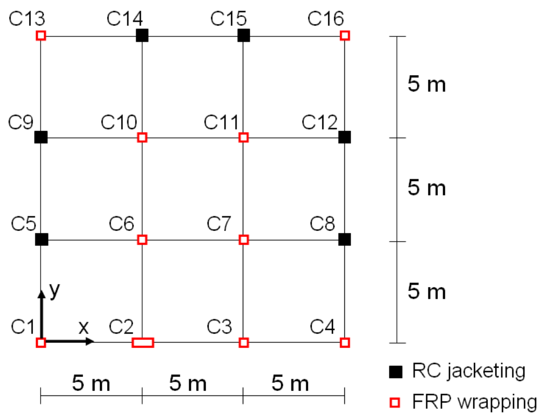

Figure 5. Schematic plan of the retrofitted structure RS1.

The capacity curves and the demand spectra for the retrofitted structure RS1 are presented in Figure 6. The retrofitting intervention reduced the irregularities of the structure and the global response could be more accurately captured by pushover analyses. Numerical outcomes pointed out that the retrofitted structure RS1 was able to withstand the displacement demand due to seismic action of Sa_g=0.29g and thus to satisfy the LSSD. In the x direction the seismic demand in terms of displacement, transformed to actual MDOF system, was reduced to 13.1 cm (15.1 cm for the bare structure), while the capacity of the structure was increased up to 13.7 cm (11.5 cm for the bare structure). In the y direction the seismic demand in terms of displacement was reduced to 13.3 cm (16.5 cm for the bare structure), while the capacity of the structure was increased up to 15.2 cm (12.8 cm for the bare structure). According to the simplified procedure based on nonlinear pushover analyses, the perimeter columns C14, C15 were detected as critical columns.

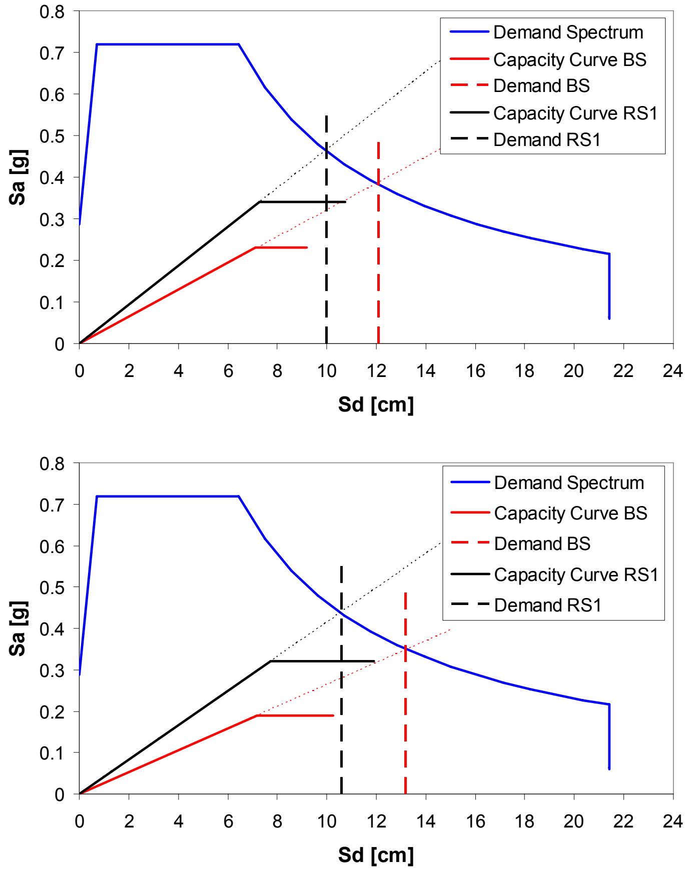

Figure 6. Demand spectra and capacity curves in AD format at LSSD ($Sa_g = 0.29$g) for the retrofitted structure RS1: x direction and y direction.

5 Nonlinear dynamic analyses

Nonlinear dynamic analyses were carried out to verify the validity of the simplified displacement-based design procedure and the effectiveness of the retrofitting intervention strategy. The suite of artificial accelerograms was generated using the computer code SIMQKE in order to match the Eurocode 8 response spectrum (Type 1, subsoil class C). Different ground motion intensities were used in the numerical analyses. Figure 7 shows the maximum roof displacement in both directions for the bare (BS) and retrofitted (RS1) models at different seismic intensity levels ranging from 0.2g to 0.4g. The maximum displacements were registered in the y direction for both the structures.

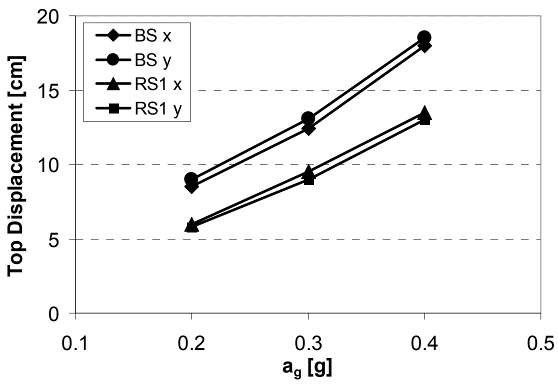

Figure 7. Maximum roof displacement in both directions for the bare and retrofitted structures as a function of the seismic intensity levels.

A comparison of the inter-storey drift profiles for the bare and retrofitted structures at Sa_g = 0.3g seismic intensity level is presented in Figure 8. The retrofitting intervention increased the stiffness of the structure and reduced the maximum inter-storey drift at all levels with respect to the bare structure. The maximum inter-storey drift was registered at the second storey for both the structures and for both the directions.

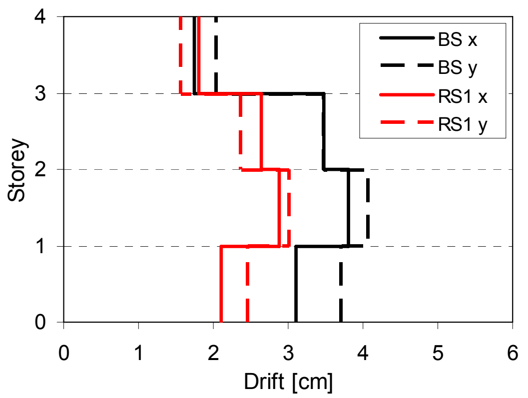

Figure 8. Storey drift profiles in both directions for the bare and retrofitted structures at Sa_g = 0.3g seismic intensity level.

Figure 9 shows the storey rotation profiles for the bare and retrofitted models at Sa_g=0.3g and Sa_g=0.4g seismic intensity levels. A considerable decrease of the storey rotation at all levels, in particular at the second level, was observed for the retrofitted structure compared to the bare counterpart. The intervention based on RC jacketing of selected columns of the structure was effective in reducing the effects of torsion and the global behaviour of the structure was improved.

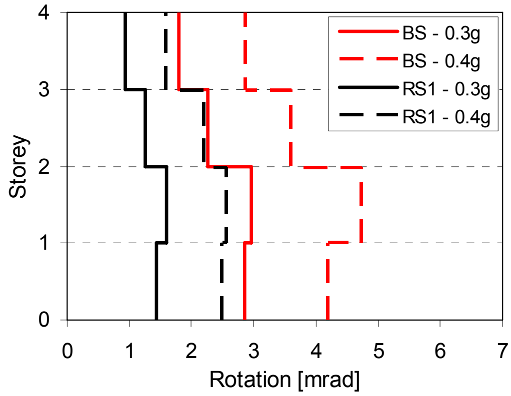

Figure 9. Storey rotation profiles for the bare and retrofitted structures at Sa_g=0.3g and Sa_g=0.4g seismic intensity levels.

The demand-to-capacity ratio (DCR), i.e. the ratio of the chord rotation demand to the chord rotation capacity, was used to evaluate the damage level of columns. The maximum chord rotation demand was obtained by numerical analyses and the chord rotation capacity was computed according to Eurocode 8 Part 3. The comparison of the maximum DCR values was carried out considering also another retrofitted configuration, named as "RS2". The structure RS2 was strengthened by using only RC jacketing for the same columns as the retrofitted structure RS1, without applying FRP wrapping to the remaining columns. Figure 10 provides the maximum DCR values registered for the columns of each storey of the bare and retrofitted models under ground motion intensity of Sa_g=0.3g. For all the models, the maximum DCR values were computed for the columns of the second storey. A significant reduction of the DCR values was observed for the columns of both the retrofitted structures.

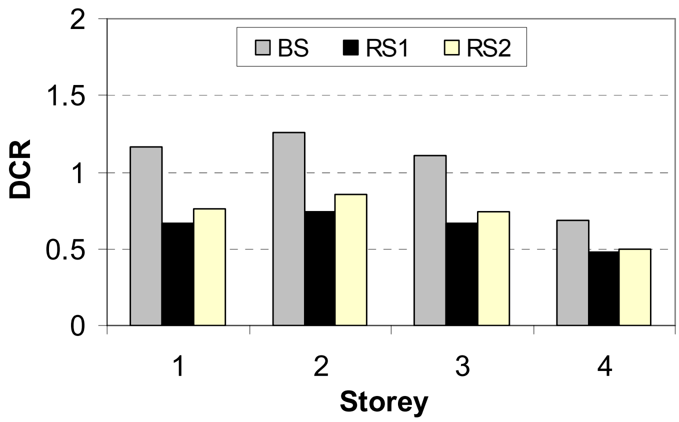

Figure 10. Maximum DCR values for the columns of each storey of the three investigated structures at Sa_g = 0.3g seismic intensity level.

Figure 11 shows the maximum DCR values for all the columns of the second storey. The maximum DCR value was registered for column C14 of the bare structure. For the retrofitted structure RS1 and RS2, the maximum DCR value was computed for column C2 and column C11, respectively. The results reported for the models RS1 and RS2 confirm the effectiveness of the retrofitting intervention. Smaller values of deformation demand were registered for the columns of the retrofitted structures compared to the bare model. Moreover, in case of model RS1, the remaining columns were detailed for ductility due to high level of confinement provided by FRP wrapping. A considerable improvement in deformation capacity was obtained and a significant decrease of the DCR values was observed for the retrofitted model RS1.

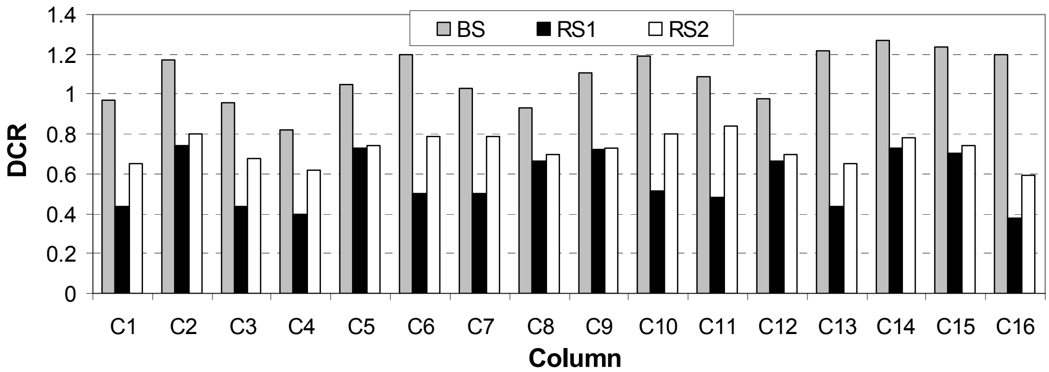

Figure 11. Maximum DCR values for the columns of the second storey of the three investigated structures at Sa_g = 0.3g seismic intensity level.

6 Conclusions

The seismic performance of a non-ductile plan-wise irregular RC structure was investigated by means of numerical analyses. A simplified procedure based on nonlinear static pushover analyses was used to select the seismic retrofitting intervention. Appropriate correction factors were computed in order to take into account the effects of torsion arising from the asymmetry of the investigated structure. The retrofitting design strategy was aimed at both reducing the torsional component of the seismic response and improving the local and global ductility of the structure. The combination of the two approaches applied to selected columns significantly improved the seismic performance of the structure. The retrofitting intervention increased the stiffness of the structure and reduced the maximum storey drift and storey rotation. Demand-to-capacity ratio values were used to evaluate the damage level of columns and to identify the most critical columns affecting the seismic performance of the structure. In the original configuration high DCR values were registered for the columns located at the flexible edges (higher demands) and for the columns with higher axial loads (lower rotational capacity). A considerable decrease of the DCR values was observed for the retrofitted model, because the deformation demand was reduced and the columns were detailed for ductility arising from the high level of confinement provided by FRP wrapping.

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