ALTERNATIVE SEISMIC RETROFIT SOLUTIONS FOR A NON-DUCTILE R/C STRUCTURE

Abstract

This study deals with the seismic assessment and retrofitting of existing under-designed R/C structures and the issue of selecting effective retrofit solutions is discussed. A displacement based procedure using nonlinear static pushover analyses is adopted in order to assess the main characteristics of the original structure and to select and compare the available retrofitting strategies. Different capacity curves can be estimated assuming different retrofitting solutions. The procedure is applied to a four-storey R/C frame tested at the JRC ELSA laboratory some years ago. The R/C frame was designed mainly for gravity loads without specific earthquake-resistant provisions. Accurate numerical models are developed to reproduce the seismic response of the R/C frame in both the original and retrofitted configurations. The effectiveness of three different retrofitting strategies is examined. First, a retrofitting intervention based on both FRP wrapping and R/C jacketing applied to selected critical columns is proposed with the main aim of improving the global ductility of the frame. Then, a retrofitting solution involving the introduction of eccentric steel bracings is investigated in order to reduce the displacement demand and to increase the energy dissipation capacity of the frame. The third intervention is carried out by adding a concrete shear wall to the short bay of the frame. This solution is efficient in controlling global lateral drift and thus reducing damage in structural members. Nonlinear dynamic analyses are performed to assess and compare the seismic response of the frame in the original and retrofitted configurations.

Keywords: seismic retrofitting, FRP wrapping, R/C jacketing, steel bracing, infill wall.

1. Introduction

This study presents the results of numerical investigations on the seismic assessment and retrofitting of a reinforced concrete (R/C) structure, designed mainly for gravity loads without specific earthquake-resistant provisions, that was pseudo-dynamically tested at the JRC ELSA laboratory in Italy. Detailed numerical models were developed to reproduce the seismic response of the R/C structure in the original and retrofitted configurations. The seismic assessment procedure was based on a simplified approach using nonlinear static pushover analyses. A detailed description of the assessment procedure used to evaluate the seismic performance of the bare structure is presented and the most relevant results are discussed. Three alternative retrofitting solutions proposed for the R/C structure are described. The first intervention scheme was defined on the basis of the results of the numerical analyses on the bare frame, adopting two different design strategies, like strength-only and ductility-only solutions. These selective interventions were applied to different members of the structure in order to improve its global and local seismic behaviour. A strength-only intervention using R/C jacketing was implemented in the strong column at the third and fourth storeys of the structure to reduce the large difference in terms of flexural capacity. Moreover, a ductility-only intervention was accomplished at the first three storeys of the structure, where a large inelastic deformation demand was expected. The intervention required the arrangement of external FRP wrapping, which provided an increase in concrete confinement of the columns. The second retrofitting intervention was based on the addition of eccentric steel bracing, that can be considered as a very effective method for global strengthening of buildings. The use of eccentric steel bracings in the rehabilitation of existing R/C structures is efficient in limiting inter-storey drifts and can provide a stable energy dissipation capacity. The third intervention was carried out by casting a concrete shear wall to the full width of the short bay of the frame. This solution leads to significant increases in overall strength and stiffness for the retrofitted frame, when compared to that of the initial frame configuration. This intervention is efficient in controlling global lateral drift and thus reducing damage in structural members, but it presents a drawback related to the adequacy of the existing foundation system to resist the increased overturning moment. Therefore the existing foundation system needs to be checked before proceeding to this type of intervention. The efficacy of the three retrofitting solutions adopted for the R/C structure was evaluated by nonlinear dynamic analyses.

2. Retrofitting strategies

A displacement based procedure is presented for the seismic assessment and retrofitting of existing R/C structures and the issue of the retrofitting strategy selection is covered. The procedure is based on a simplified approach using nonlinear static pushover analyses and allows to compare alternative retrofitting strategies countering different structural deficiencies. The base shear-top displacement relationships were transformed into capacity curves in the acceleration-displacement (AD) format. Several target displacements and capacity curves were estimated assuming that different strategies could be used to retrofit the structure. Different types of structural interventions on existing structures can be classified according to their effect on the behaviour of the structure. Each type of structural intervention can be represented by the capacity curves presented in Figure 1. The group A includes strategies whose basic aim is to improve the overall ductility of the structure; the group B represents interventions resulting in system strengthening and stiffening; the intermediate group (group C) comprises solutions increasing both strength and ductility of the structure. The range of available retrofit solutions is bounded by assuming the two alternative structural intervention techniques corresponding to group A and B.

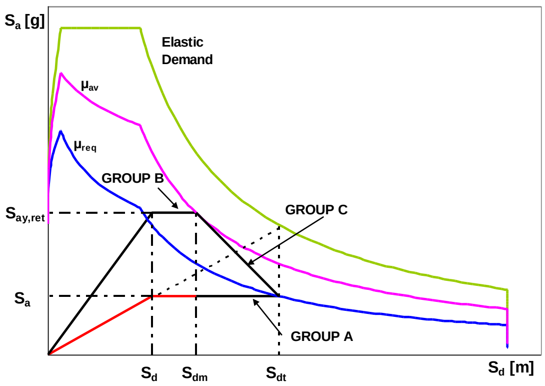

Figure 1. Capacity curves for different retrofitting strategies

3. R/C structure under investigation

The seismic assessment and retrofitting procedure based on a displacement based approach was applied to a four-storey R/C frame, designed only for gravity loads, that was subjected to real-scale pseudo-dynamic tests at the JRC ELSA laboratory at Ispra. The frame was intended to be representative of the typical design and construction practice in many Southern European countries in the 1960's. It was also representative of a number of rather recent buildings, designed without the application of the capacity design principles and without up-to-date detailing. The elevation of the R/C frame, the geometric characteristics and the reinforcement details of the columns are shown in Figure 2. A more detailed description of the structure can be found in a JRC ELSA Report, Pinto (2002). The full-scale R/C frame was modelled by using both the code SeismoStruct, based on a fibre-modelling approach, and the code Ruaumoko, based on a lumped plasticity approach. The actual materials properties measured during the experimental tests were introduced into the numerical models. The comparison of numerical predictions with experimental test results allowed to calibrate the main parameters of the developed numerical models.

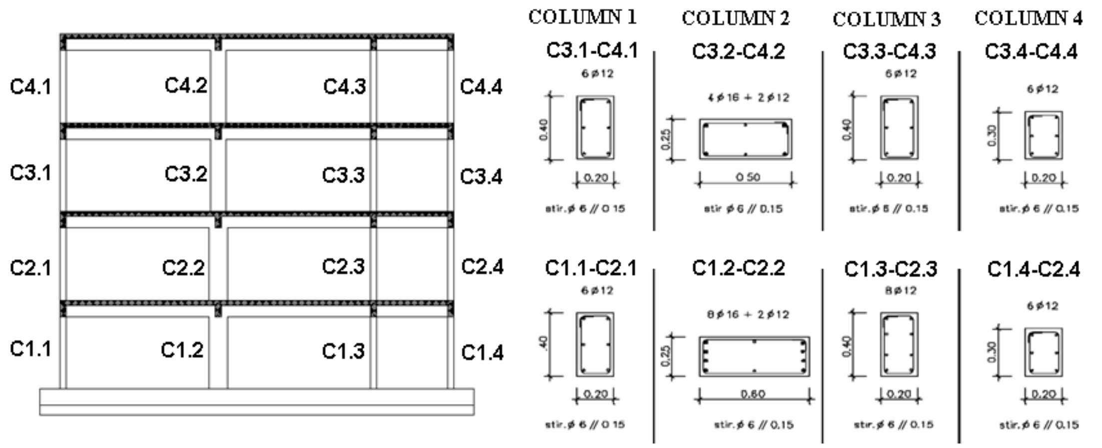

Figure 2. Geometric characteristics and reinforcement details of the columns of the test structure

4. Simplified assessment procedure for the bare structure

Nonlinear static pushover analyses were performed to obtain the capacity of the structure represented by a force-displacement curve. The base shear force and the roof displacement were converted to the spectral accelerations and to the spectral displacements of an equivalent SDOF system, and these spectral values defined the capacity spectrum. The bilinear (elastic-perfectly plastic) idealization of the pushover curve was defined on the basis of the "equal-energy" concept (the areas underneath the actual and bilinear curves are approximately the same, within the range of interest). In Figure 3 the seismic demand for the equivalent SDOF system was determined for the Limit State of Significant Damage (LSSD). The elastic acceleration and the corresponding elastic displacement demand were computed by intersecting the radial line corresponding to the elastic period of the idealized bilinear system with the elastic demand spectrum. The inelastic demand in terms of accelerations and displacements was provided by the intersection point of the capacity curve with the demand spectrum corresponding to the ductility demand. In this study, the seismic demand was computed with reference to the Eurocode 8 response spectrum (Type 1, soil type A). The theoretical predictions were performed for a PGA level equal to 0.3g. The value of the total ultimate chord rotation capacity, ϑ_u, of concrete members under cyclic loading may be calculated from the following expression, according to Eurocode 8:

$$\theta_u = \frac{1}{\gamma_{el}} \cdot 0.016 \cdot \left(0.3^{\nu}\right) \cdot \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} \cdot f_c\right]^{0.225} \cdot \left(\frac{L_V}{h}\right)^{0.35} \cdot 25^{\left(\alpha \cdot \rho_{sx} \cdot \frac{f_{yw}}{f_c}\right)} \cdot (1.25^{100 \cdot \rho_d}) \quad (1)$$

The Limit State of Significant Damage (LSSD) corresponds to the attainment of $0.75 \cdot \theta_u$. Figure 3 shows that the bare frame lacked the appropriate capacity to resist the 0.3g PGA seismic intensity level at the LSSD. The displacement demands in Figure 3 refer to the equivalent SDOF system. The displacement demands of the MDOF system were obtained by multiplying the SDOF system demand by the transformation factor $\Gamma = \sum m_i \phi_i / \sum m_i \phi_i^2$, where m_i is the mass in the i^{th} storey and ϕ_i is the component of the normalized displacement shape. A gap in terms of maximum top displacement was observed at the LSSD; the difference between the seismic demand and the displacement capacity was 0.036m (0.098m vs 0.062m). The results of the simplified procedure showed that the attainment of the LSSD occurred at column 2 of the third floor, where the most significant damage was observed in the laboratory tests and the highest value of the Demand-to-Capacity Ratio was registered during nonlinear dynamic analyses, see Figure 7. The developed numerical models provided reliable predictions of the behaviour of the test specimen identifying the main structural deficiencies.

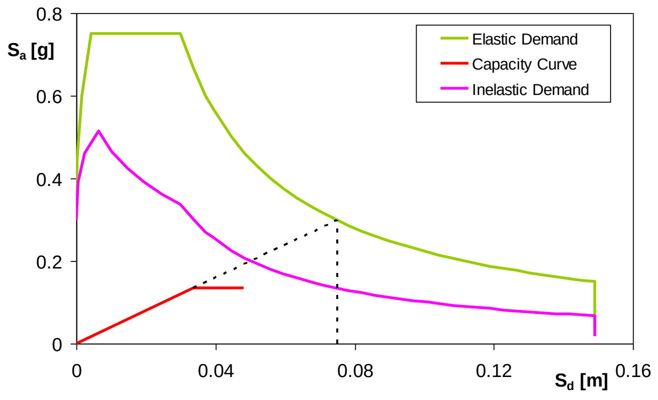

Figure 3. Demand spectra and capacity curve in AD format at the LSSD for the bare frame

5. Selective retrofitting intervention using FRP wrapping and R/C jacketing

A selective retrofitting intervention using both glass fiber-reinforced polymer (FRP) laminates and R/C jacketing was considered in order to allow the frame to withstand 0.3g PGA seismic actions. This solution was not experimentally carried out and only numerical simulations were performed. The retrofitting intervention was aimed to achieve a more ductile global performance of the frame by increasing the ductility of the columns and by preventing brittle failure modes. The aims of this retrofitting intervention were: 1) to prevent the soft-storey mechanism at the third floor by mitigating the strength difference involved by the change of section of the strong column between the second and the third floor; 2) to improve the global deformation capacity of the frame by increasing the ductile resources of the columns, preventing brittle failure modes. According to a selective retrofitting scheme, a ductility-only intervention was applied to the first three storeys and a strength-ductility intervention was applied to the strong column at the top two storeys, Figure 4. A mixed intervention was carried out using FRP, whose principal characteristics are reported in Table 1, and R/C jacketing. The FRP consisted of a 3-layer wrapping applied to the columns of the first three floors; the R/C jacketing consisted of 4+4 steel bars on two opposite sides of the strong column at the top two storeys.

A numerical model of the frame retrofitted with FRP wrapping and R/C jacketing was developed by using the SeismoStruct code. The nonlinear variable confinement model that includes the constitutive relationship and cyclic rules proposed by Mander (1988) in compression, and those of Yankelevsky and Reinhardt (1989) in tension, was adopted to model R/C sections retrofitted by FRP. The confinement effect introduced by the FRP wrapping was modelled by means of the rules proposed by Spoelstra and Monti (1999). The model is based on modified Mander’s model (Mander, 1988), where maximum strength and corresponding strain of the confined concrete are defined as a function of confinement pressure. A constant lateral pressure, depending on steel yielding stress, is considered for steel confinement, whereas confinement pressure is linearly varying with concrete lateral dilation in the case of external FRP wrapping. An iterative procedure is adopted to obtain the axial stress corresponding to a given value of axial strain, taking into account the effect of confinement, according to Spoelstra and Monti (1999). The R/C jacketed rectangular section available in Seismostruct libraries was used for the modelling of rectangular columns retrofitted by means of R/C jacketing. Different confinement levels for the internal (pre-existing) and the external (new) concrete materials were defined. To evaluate the properties of the retrofitted elements, the following assumptions were adopted according to Eurocode 8: the jacketed column behaves monolithically with full composite action between old and new concrete, the concrete properties of the jacket apply over the full section of the element, the axial load is considered acting on the full composite section.

The enhancement of the deformation capacity of the member, ϑ_u, was determined by adding a term due to FRP to the term describing the confinement provided by the transverse reinforcement. The total chord rotation capacity was calculated from expression (1) with the exponent of the term due to confinement increased by ($\alpha^* \rho_f \; f_{f,e} / f_c$), where:

- $\alpha^* = 1 - \dfrac{(b-2R)^2 + (h-2R)^2}{3bh}$ is the confinement effectiveness factor, where R = 20 mm is the radius of the rounded corner of the section and b, h are the full cross-sectional dimensions,
- $\rho_f = \dfrac{2t_f}{b}$ is the FRP ratio parallel to the loading direction,
- $f_{f,e} = \min\left(f_{u,f};\varepsilon_{u,f}E_f\right)\left(1-0.7\cdot\min\left(f_{u,f};\varepsilon_{u,f}E_f\right)\dfrac{\rho_f}{f_c}\right)$ is an effective stress, where $f_{u,f}$ and E_f are the strength and the elastic modulus of the FRP and $\varepsilon_{u,f}$ is the ultimate strain.

The capacity curve and the demand spectra for the equivalent SDOF system are presented in Figure 5, which shows that the retrofitted frame was able to satisfy the LSSD. The seismic demand in terms of displacement, transformed to actual MDOF system, was equal to 0.094 m, while the capacity of the frame was increased up to 0.104 m (0.062 m in the bare frame). The retrofitted frame fully complied with the LSSD requirements, in contrast with the response of the original frame which lacked the required ductility. The soft-storey failure mechanism was eliminated, allowing the LSSD performance target to be met. Column confinement generated by the application of FRP provided the frame with significantly enhanced ductility and allowed it to achieve the seismic demand by increasing the plastic branch of the base shear - top displacement curve. The retrofitting intervention increased slightly the stiffness and strength of the structure and increased considerably its global deformation capacity. The adopted selective scheme proved to be effective for seismic upgrading of the R/C frame.

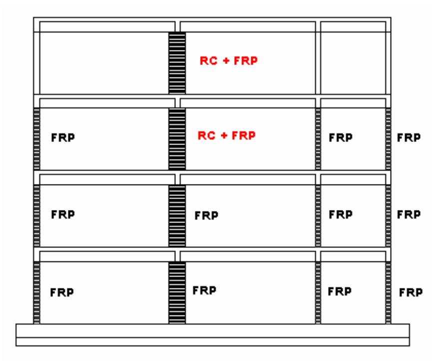

Figure 4. Selective retrofitting intervention with FRP wrapping and R/C jacketing

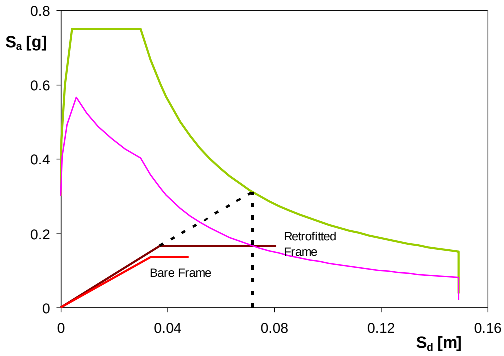

Figure 5. Demand spectra and capacity curve in AD format at the LSSD for the frame retrofitted with FRP wrapping and R/C jacketing

Table 1. Characteristics of GFRP used for the retrofitting intervention

Young Modulus (GPa)	Ultimate Tensile Stress (MPa)	Ultimate Tensile Deformation	Layer thickness (mm)
65	1700	0.026	0.23

Non linear dynamic analyses by using seven artificial accelerograms with PGA=0.3g were carried out to verify the validity of the simplified procedure and the effectiveness of the retrofitting intervention. The suite of artificial accelerograms was generated using the computer code SIMQKE so as to match a Type 1 response spectrum for soil class A, according to Eurocode 8. The storey drift profile of the retrofitted model exhibited an increase of the drift value at the first two floors, while a significant reduction of the storey drift occurred at the third floor, Figure 6. The increase of the flexural strength of column 2 and the confining effect of the FRP prevented the development of a soft-storey behaviour at the third floor. Figure 7 provides the values of the Demand-to-Capacity Ratio in terms of chord rotation obtained from the dynamic analyses for the columns of the bare and retrofitted models. A significant reduction of the Demand-to-Capacity Ratio can be observed for all the columns of the first three floors, above all at the third floor. Numerical analyses indicated high values of deformation demand in the strong column, but in this case the column was detailed for ductility due to high level of confinement provided by FRP. A considerable improvement in deformation capacity was obtained and a significant decrease of the Demand-to-Capacity Ratio was observed for the retrofitted model.

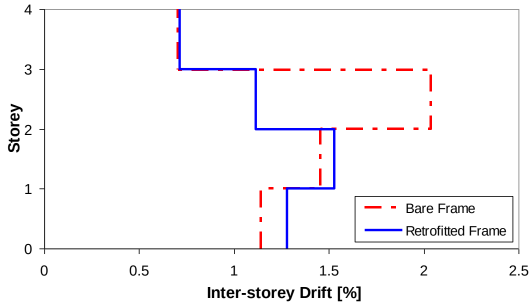

Figure 6. Inter-storey drift profiles for the bare and retrofitted frames

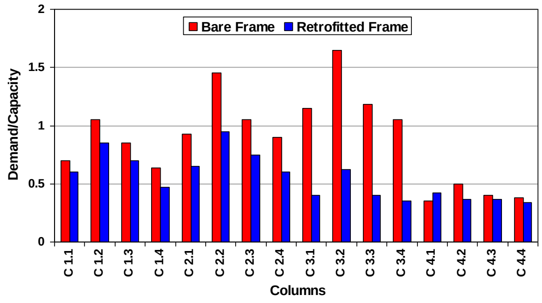

Figure 7. Demand-to-Capacity Ratio for the bare and retrofitted frames

6. Retrofitting intervention using steel eccentric bracing

A retrofitting solution involving the introduction of eccentric steel bracing (chevron bracing with a vertical shear link) was also investigated. The steel bracing assembly was inserted in the middle bay of all the floors. The vertical shear link was located at the midspan underneath the floor beams and connected to a chevron bracing. For the link specimen an European HE120A section was adopted. Vertical steel straps were connected to the adjacent columns and two horizontal steel beams were anchored to the floor beams. Figure 8 shows a schematic view of the proposed eccentric bracing system inserted in the middle bay of the R/C frame. A link model was developed for the numerical analyses of the retrofitted frame, based on the approach proposed by Ricles and Popov (1987) for horizontal shear link elements. Steel links are subjected to high levels of shear forces and bending moments in the active link regions and elastic and inelastic deformations of both the shear and flexural behaviours have to be taken into account. The link was modelled as a linear beam element with nonlinear rotational and translational springs at the ends. The rotational spring was used to represent the flexural inelastic behaviour and the translational springs were used to represent the inelastic shear behaviour of the link web. Multilinear relationships were assumed for the shear force-deformation and bending moment-rotation curve. Isotropic hardening was used in shear yielding, while kinematic hardening was assumed for moment yielding, as suggested by experimental evidence (Kasai and Popov, 1986; Ricles and Popov, 1987). An upper bound of the shear force after complete hardening was considered. The shear link model was implemented in the numerical models of the frame developed by the computer code Ruaumoko. The accuracy and reliability of the developed shear link model was verified through comparison with experimental tests performed at the JRC ELSA laboratory.

Nonlinear static pushover analyses were performed on the retrofitted frame in order to estimate the effectiveness of the applied retrofitting technique on the global structural behaviour. The results in terms of maximum top displacement required for 0.3g PGA level are reported in the AD format in Figure 9. The retrofitting intervention considerably increased the stiffness and strength of the structure. The retrofitted frame was able to satisfy the LSSD and the capacity exceeded the demand. The seismic demand in terms of displacement, transformed to actual MDOF system, was equal to 0.06 m (0.098 m for the bare frame), while the capacity of the frame was increased up to 0.071 m (0.062 m in the bare frame). The procedure confirmed the effectiveness of the retrofitting intervention in both reducing the displacement demand and in increasing the global deformation capacity of the bare frame. The retrofitting intervention eliminated the irregularities of the frame and the global response of regular structures may be captured more accurately by pushover analyses.

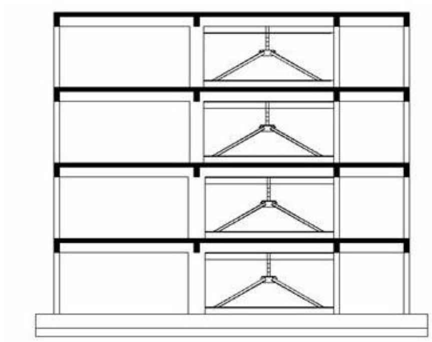

Figure 8. Elevation view of the R/C frame retrofitted with eccentric steel bracing

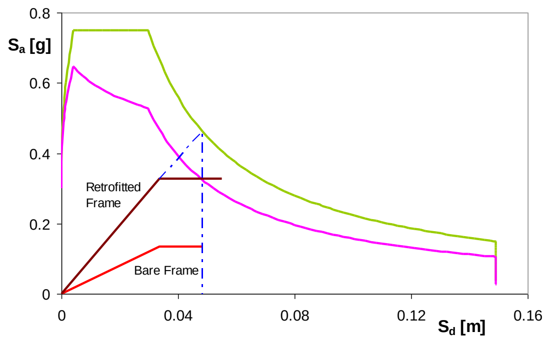

Figure 9. Demand spectra and capacity curve in AD format at the LSSD for the frame retrofitted with eccentric steel bracing

Nonlinear dynamic analyses were carried out using seven artificial accelerograms with PGA=0.3g, as described in the previous paragraph. Significant decreases of the values of the inter-storey drift and of the Demand-to-Capacity Ratio were registered in the strengthened frame, in particular at the third storey, Figures 10 and 11. This confirms the effectiveness of the retrofitting intervention in increasing the strength and stiffness of the structure and in preventing the formation of the soft-storey mechanism at the third floor. The maximum value of the Demand-to-Capacity Ratio for the bare frame was registered in the strong column at the third storey; for the strengthened frame the highest values of the Demand-to-Capacity Ratio were found in the columns of the first two storeys.

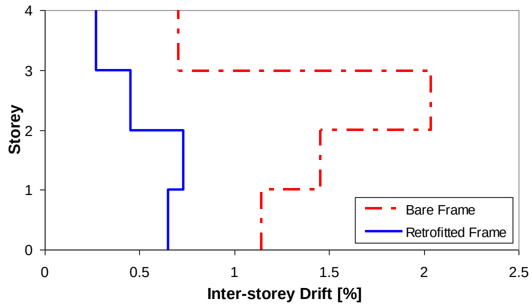

Figure 10. Inter-storey drift profiles for the bare and retrofitted frames

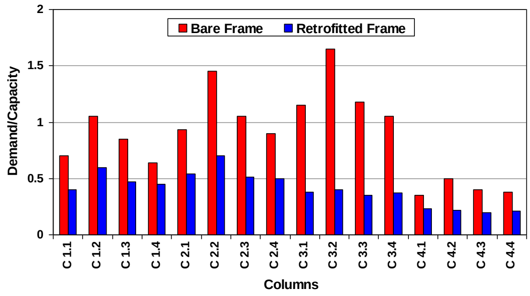

Figure 11. Demand-to-Capacity Ratio for the bare and retrofitted frames

7. Retrofitting intervention using infill wall

The third intervention proposed in this study is one of the most common method used for strengthening existing R/C structures and consisted in adding a concrete shear wall to the frame. The infill wall was introduced into the short bay of the frame by incorporating existing columns C3 and C4. The existing columns were not adequate to act as boundary elements and therefore additional boundary elements were created. As regards the connection between the existing members and the infill wall, dowels were placed between the wall and the columns in order to assure a monolithic connection. The elevation of the retrofitted frame and the cross-section of the wall with reinforcement detailing are shown in Figure 12. The height of the critical region was equal to the height of the first storey. Two structural systems were identified in the building: the wall and the other vertical elements. The behaviour of the retrofitted frame was significantly influenced by the presence of the wall. Nonlinear static analyses were performed on the retrofitted frame and the base shear – top displacement curve was obtained. The calculated response of the frame for the case of adding an infill wall in the short bay is compared with the response of the original frame in Figure 13. The insertion of the infill wall increases substantially the stiffness and strength of the original frame. The base shear of the wall retrofit solution is about four times greater than the base shear of the bare frame and 1.65 times greater than the base shear of the bracing retrofit solution.

The effectiveness of the wall retrofit solution was assessed by performing nonlinear dynamic analyses with the same records as the previous structural configurations. The inter-storey drift profile imposed by artificial accelerograms with PGA=0.3g is presented and compared with the inter-storey drift profile of the bare frame, Figure 14. A considerable reduction of inter-storey drifts was registered for all the levels. The wall acted as a stiff vertical spine preventing the formation of a soft-storey mechanism and the maximum drift was smaller than the case of the bracing retrofit solution. The maximum value of the Demand-to-Capacity Ratio for the retrofitted frame was registered for the wall, Figure 15. Due to the large cross-section dimensions of the wall, the deformation capacity in terms of chord rotation was smaller than that of slender columns. A drastic reduction of deformation demands was observed for all the other columns.

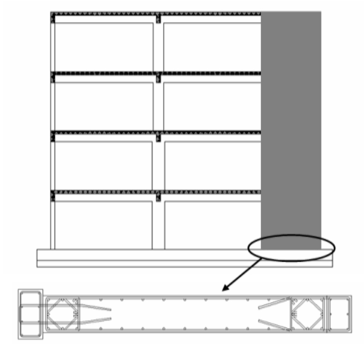

Figure 12. Elevation view of the retrofitted R/C frame and cross-section detailing of the infill wall

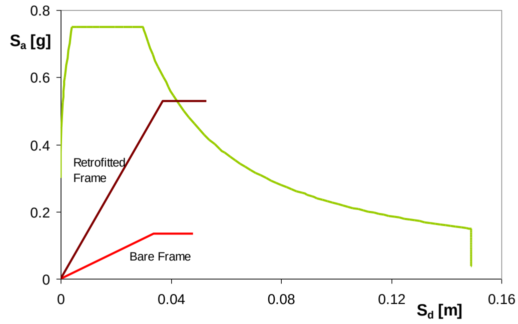

Figure 13. Demand spectra and capacity curve in AD format at the LSSD for the frame retrofitted with infill wall

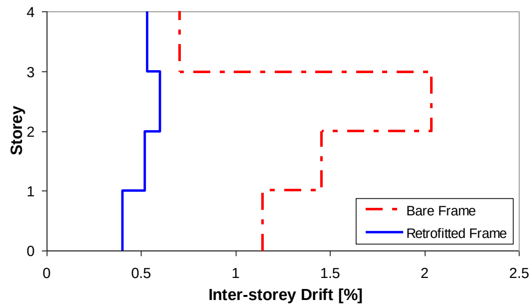

Figure 14. Inter-storey drift profiles for the bare and wall-retrofitted frames

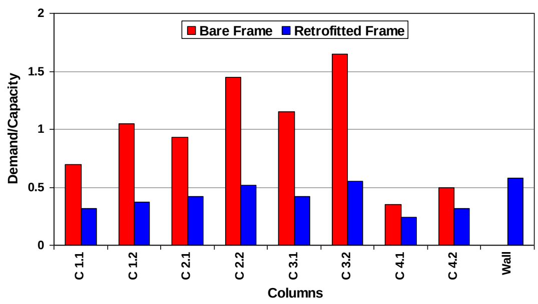

Figure 15. Demand-to-Capacity Ratio for the bare and wall-retrofitted frames

8. Conclusions

The aim of this study was to investigate a simplified displacement based procedure for the seismic assessment and retrofitting of a four-storey R/C frame, designed mainly for gravity loads without specific earthquake-resistant provisions and tested at the JRC ELSA laboratory. The effectiveness of three retrofitting interventions with different aims was assessed as well. Detailed numerical models of the R/C frame in the bare and retrofitted configurations were developed and static pushover analyses were performed. The theoretical predictions of the simplified procedure in terms of global performance showed that the bare frame was unable to satisfy the demand due to the 0.3g PGA level at the Limit State of Significant Damage. A selective retrofitting intervention was modelled and investigated for the enhancement of the seismic performance of the R/C frame. The FRP wrapping increased the deformation capacity of the columns and the R/C jacketing was found to be effective for mitigating the abrupt change in the flexural capacity of the strong column at the third floor and for avoiding the soft-storey failure mechanism. The second retrofitting intervention proposed in this study and based on the introduction of eccentric steel bracings reduced the displacement demand on the frame and increased the energy dissipation capacity of the system. The third retrofit solution based on the addition of a concrete shear wall was extremely efficient in controlling global lateral drift and thus reducing damage in frame members. However the drawback of this method is related to the strengthening of the existing foundation system to resist the increased overturning moment. The drawbacks and economic losses related to the addition of walls may render the local retrofit solution carried out by using R/C jacketing or FRP wrapping more appealing. Numerical results obtained from the simplified displacement based procedure were confirmed by nonlinear dynamic analyses.

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