Structural Engineering and Mechanics, *Vol. 73, No. 1 (2020) 53-64*
DOI: https://doi.org/10.12989/sem.2020.73.1.053

Case study on seismic retrofit and cost assessment for a school building

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(Received July 25, 2018, Revised March 27, 2019, Accepted August 27, 2019)

Abstract. In different high seismic regions around the world, many non-ductile existing reinforced concrete frame buildings, built without adequate seismic detailing requirements, have been damaged or collapsed after past earthquakes. The assessment and the retrofit of these non-ductile concrete structures is crucial theme of research for all the scientific community of engineers. In particular, a careful assessment of the existing building is fundamental for understanding the failure mechanisms that govern the collapse of the structure or the achievement of the recommended limit states. Based on the seismic assessment, the best retrofit strategy can be designed and applied to the structure. A school building located in Avellino province (Italy) is the case study. The analysis of seismic vulnerability carried out on the mentioned building has highlighted deficiencies in both static and seismic load conditions. The retrofit of the building has been designed based on different retrofit options in order to show the real retrofit design developed from the engineers to achieve the seismic safety of the building. The retrofit costs associated to structural operations are calculated for each case and have been summed up to the costs of the in situ tests. The paper shows a real retrofit design case study in which the best solution is chosen based on the results in terms of structural performance and cost among the different retrofit options.

Keywords: seismic retrofit, RC jacketing, steel jacketing, FRP wrapping, retrofit costs

1. Introduction

The retrofit of existing structures in highly seismic zones is one of the most challenging issue in earthquake engineering. In fact, post-earthquake reconnaissance and recent research on seismic risk analysis have shown that non-ductile concrete frame structures are much more susceptible to collapse than modern code-conforming frames. The assessment and the retrofit of these non-ductile concrete structures is crucial theme of research for all the scientific community of engineers. In particular, a careful assessment of the existing building is fundamental for understanding the failure mechanisms that govern the collapse of the structure or the achievement of the recommended limit states. Based on the seismic assessment, the best retrofit strategy can be designed and applied to the structure. Many conventional retrofit methods, such as concrete or steel jacketing of the columns, addition of shear walls and methods often based on new materials as fiber reinforced polymers (FRP), have been proposed and used (Moehle 2000, and Thermou and Elnashai 2006, Proenca and Gago 2011, Calvi 2013, Guneyisi and Azez 2016, Saribiyik and Caglar 2016, Formisano *et al*. 2017, Miano *et al*. 2018a, 2019a). These methods can be applied considering the desired limit states/performance levels, using the requirements of new seismic codes or more advanced performance based approaches to measure the probability of collapse and quantify and minimize the costs and/or the losses with different approaches (Aslani 2005, Goulet *et al*. 2007, Liel and Deierlein 2013, Jalayer *et al*. 2015, Jalayer and Ebrahimian 2017, Jalayer *et al*. 2017, Miano *et al*. 2018b, 2019b). Non-linear static analysis procedure, also known as pushover, has been implemented in this work for seismic and static safety assessment and for measure the effectiveness of the different retrofit strategies. In particular, pushover analysis, can be used to calculate the vulnerability index, also called seismic safety factor (Frascadore *et al*. 2015) and defined in NTC 2018.

The main goal of the retrofit design is to prevent premature failure of brittle elements and to increase their ductility and strength. In addition, the lateral displacements need to be distributed relatively uniformly over the height of the structure to avoid concentration of inelastic deformations in a story mechanism. To control lateral drifts and to keep them below the target displacement, one of the effective strategies for moment frame concrete structures is to add lateral stiffness, e.g., by adding a shear wall, to reduce the period and decrease the resulting building displacements. Another effective way to increase overall ductility and strength of the frame is to increase flexural and shear strength and additional deformation capacity of individual lateral load resistant members. This can be achieved by better confining the columns and shifting the failure mode from brittle shear to ductile flexural mode. Enlarging the cross section of concrete jacketing can increase lateral stiffness, strength, and ductility.

The selection of the retrofit scheme and the level of intervention is a rather complex process because many factors need to be considered. To avoid and reduce the restriction of use of building for a long time, addition of new lateral load resistance system, member replacement or

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http://www.techno-press.com/journals/sem&subpage=7
ISSN: 1225-4568 (Print), 1598-6217 (Online)

local or global modification (addition of stiffness, strength and ductility) of elements and system may be difficult or impossible to implement. The ease and quick application of fiber reinforced polymers (FRP) composite materials makes them attractive for use in structural applications, especially in cases where dead weight, space or time restrictions exist.

In addition to the assessment of the structural performance, also the costs related to each of the retrofit option are evaluated in order to show all the design process really developed for the seismic retrofit of the case study building. The retrofit structural costs related to structure in elevation of the buildings (excluding roof and foundations retrofit operations) have been calculated for each case and have been summed up to the costs of the in situ in tests to have a full cost of the retrofit design project. The costs should be weighted with the improvement of the threshold of the seismic safety, measured in terms of the vulnerability index.

A school building located in Avellino province (Italy) is the case study analyzed in this work. The analysis of seismic vulnerability carried out on the mentioned building has highlighted deficiencies in both static and seismic load conditions. The retrofit of the building has been designed based on different retrofit options in order to show the real retrofit design developed from the engineers to achieve the seismic safety of the building. The 3D model of the building has been produced using the commercial software for structural calculation CDS 2018.

Different retrofit options are considered: 1) all columns and beams of the building are steel jacketed, 2) all columns and beams of the building are FRP wrapped, 3) all columns of the frame are RC jacketed and all the beams are FRP wrapped and 4) all columns of the frame are RC jacketed and all the beams are steel jacketed. If possible, the same retrofit option has been designed more times in order to achieve different levels of the vulnerability index (0.6 and 0.8 of the seismic safety and the complete seismic safety, Frascadore *et al*. 2015, NTC 2008 and NTC 2018). For some retrofit options, it is impossible to achieve all the different levels of the vulnerability index, due to some technological limitations (e.g., there is a maximum number of FRP sheets that can be applied etc.).

2. Existing building assessment

2.1 Description of the building

The case study building hosts a primary school in the district of Avellino (Italy). It was built in the '60s in various phases with a reinforced concrete frame structure and without adequate seismic detailing requirements. The building consists of one level below the ground, two levels above ground and the roof. The roof is for the most part not practicable, except for a storage room and a small buried portion (formerly disused boiler room). It should be noted that the first level is placed at about 2.50 m from the foundation floor, represented by isolated plinths. The structural interstorey height among floors is about 3.60 m. Geometrically, the roof has inclined slopes covered by tiles. At the last storey, the maximum interstorey height measured

Fig. 1 Photo of the lateral view of the building

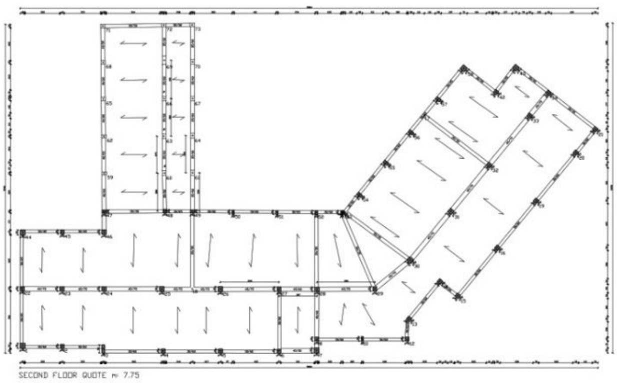

Fig. 2 Typical floor plan of the building

at the intrados of the roof ridge is about 2.70 m. Apart from the external emergency staircase, there is only one internal staircase. The one way slabs are made of precast reinforced concrete beams and blocks. The building turns out to be irregular, because the first two floors have an almost L-shaped plan, whereas the other two floors have an approximately rectangular plan layout. The school building area is around 980 m^2 at the 1^{st} and 2^{nd} floors, while it is around 810 m^2 at the 3^{rd} and 4^{th} floors. Figure 1 and Figure 2 show respectively a photo and a typical plan of the building.

As shown in the plan, the most recurring sections for the columns in the first and second floors are 45x50 cm and 40x40 cm with 3Ø16 plain bars per side and plain ties Ø8/25 cm. Instead, for the other two floors, the sections are 40x40cm and 30x30 cm with 2Ø16 smooth bars per side and plain ties Ø8/30 cm. The beams sections are 45x75 cm and 40x60 cm reinforced with 4Ø16 plain bars inferiorly, with the minimum of bars superiorly (according to the common procedure of the construction age of the building) and with plain stirrups Ø8/30 cm. Moreover, it is to note the presence in the basement of three columns that are not present in any of the other floors. Their function is related only to avoid excessive deformations in the upper slab.

2.2 In situ tests investigations for knowledge level 2

The process of the knowledge, regarding the definition of structural system, seismic details presence and mechanical material properties, is the most important and crucial point in the assessment of an existing building, particularly in a highly seismic zone such as for the case study. A plan of diagnostic investigations on the structural and non-structural elements has been executed from the engineers. In fact, the opportunity to have the school building partially available and therefore in a condition of easier invasive investigation has induced the structural engineers to build a detailed campaign of tests in order to have a more complete characterization of the existing structure.

The principal categories for testing the concrete of the structure are non-destructive and destructive methods. The non-destructive methods can be performed directly on the in-situ concrete without removal of a sample, although removal of surface finishes is likely to be necessary. The destructive tests are instead in the category of the methods that require sample extraction. Samples are most commonly taken in the form of cores drilled from the concrete, which may be used in the laboratory for strength and other physical tests as well as visual, petrographic and chemical analysis. Some chemical tests may be performed on smaller drilled powdered samples taken directly from the structure, thus causing substantially less damage, but the risk of sample contamination is increased and precision may be reduced (Bungey and Grantham 2014). Sample of steel is subjected to a wide variety of mechanical tests to measure their strength, elastic constants, and other material properties as well as their performance under a variety of actual use conditions and environments. Tensile test is the most common of them. In particular, uniaxial tensile test is known as a basic and universal engineering test to achieve material parameters such as ultimate tensile strength (UTS), yield strength (YS), % elongation, % area of reduction and young's modulus.

The types of tests performed for the case study can be summarized as follows:

a) Compression breaking of cylindrical specimens of concrete belonging to structural elements in order to evaluate their compressive strength;

b) Non-destructive tests for structural elements in order to determine the concrete compressive strength;

c) Tensile tests on steel bars;

d) Non-destructive tests for structural elements in order to determine the presence and quantity of steel in the aforementioned elements.

The number of surveys and tests on structural elements concrete and steel through destructive tests and non-destructive tests is one of the most challenging and discussed topic in the codes around all the world.

Italian code (NTC 2008 and 2018), such as European code (Eurocode 8 part 3), allows to the professional engineers to achieve one of three knowledge levels as function on the information they have about existing building material properties and as function of the number of tests on concrete and steel they plan to do. A confident factor should be applied to material properties values. This value is different based on the achieved knowledge level. Table 1 summarizes the knowledge levels definition for materials, methods of analysis and confidence factors (CF), as described in Commentary (2009). It is to be considered

Table 1 Knowledge levels definition for materials, methods of analysis and confidence factors (CF) (BS EN 1998-3:2005)

Knowledge level	Materials	Analysis	Coefficient
KL1	Default values for the time of construction plus limited in situ tests	Lateral force procedure and Modal response spectrum analysis	1.35
KL2	Original design specifications plus limited in situ tests or extended in situ tests	All	1.20
KL3	Original test reports plus limited in situ tests or comprehensive in situ tests	All	1.00

that the design project has been done before the publication of NTC 2018, so the formal reference is to NTC 2008 and to its Commentary (2009), but the principles are the same also in NTC 2018. Table 2, instead, summarizes the recommended minimum requirements for different levels of testing. It is important to note that in the Commentary (2009) allows to replace each destructive test with three non destructive tests up to the 50% of all the required destructive tests.

It is important to highlight the recommended minimum requirements for different levels of testing with reference to the definitions in Table 1 related to limited, extended and comprehensive tests. In particular, 1 sample for each 300 m^2 of floor of the building for each type of structural member is needed to achieve the level of limited in situ tests; 2 sample for each 300 m^2 of floor of the building for each type of structural member are needed to achieve the level of extended in situ tests; 3 sample for each 300 m^2 of floor of the building for each type of structural member are needed to achieve the level of comprehensive in situ tests.

The engineers have chosen to achieve the level of knowledge 2, also in order to use static non linear analysis as procedure for the assessment. Only the default values for the time of construction were available, so in order to achieve the level of knowledge 2, extended in situ tests were needed. The number of tests required is based on the dimensions in plan of the building at each floor. The floor area is around 980m^2 in the 1st and 2nd floors, while is around 810m^2 in the 3rd and 4th floors; from an engineering point of view and based on the recommendation previously described, it is assumed to have 6 samples for columns and 6 samples for beams at each floor.

Table 2 summarizes the number of tests and the average values for the concrete compressive strength for both destructive and non-destructive test (where D indicates the number of destructive tests, while ND stands for the number of non-destructive tests). Moreover, the final average values used at each floor for both columns and beams are summarized in Table 2. The final average value of the concrete compressive strength is a weighted average carried

Fig. 3 Concrete cores and steel bars extracted and tested in the case study

out for columns and beams at each floor, where the destructive tests have weight 1 while the non-destructive ones have weight 1/3. It is, however, to highlight that Italian code (NTC 2008) suggests to calculate a global average for material properties through all the floors for each structural elements. The approach proposed in the recent Italian guidelines is instead used here (Aversa *et al*. 2012). This approach says that in case of strong variability among the different floors/members with respect to mechanical material properties and in case the number of tests is consistent, it is possible to distinguish the values of the properties among the different members/floors.

A very important to discuss herein is that in the real applications/design projects sometimes is impossible to do the tests in each part of the building due to physical obstacles. In the case study, the tests have been done where possible. For example in the foundation floor, many members were covered by the terrain/ground and it was practically difficult and expensive to investigate them. In fact, the mechanical equipment was not able to work in these areas, due to the reduced spaces. Another important problems occur in the columns, where there has been the inability to take samples of steel bars on the upper floors. This was related to the fact that for all columns in these floors, it was found the presence of only the corner bars. This circumstance has been judged dangerous from the engineers. Then, Thus, steel pieces from the corner bars were not collected. However, the variability of the mechanical material properties in the steel can be considered inferior with respect to concrete (Verderame *et al*. 2001). Figure 3 shows the concrete cores and the steel bars extracted from structural members for the case study.

It appears clear that the regression used from lab test (RILEM 1993) in order to pass from the pulse velocity and rebound index (measured in ultrasonic tests) to the concrete compressive strength gives quite unreliable results. In fact, the comparison of the average values for destructive and non-destructive tests shows a strong disagreement. The authors strongly encourage to calibrate the parameters of the regression based on own in situ sample or to use different and more appropriate regressions typologies.

Table 2 No of destructive tests for beams and columns at each floor and average values for concrete compressive strength

Members	D tests	Average (MPa)	ND Tests	Average (MPa)	Weighted average (MPa)
Foundation columns	3	24.28	8	15.52	21.15
Foundation beams	5	18.85	6	13.27	17.92
Ground floor columns	14	17.12	10	9.01	15.96
Ground floor beams	4	11.79	5	8.60	11.54
First floor columns	4	11.94	13	6.06	9.42
First floor beams	4	12.09	8	5.02	10.01
Roof columns	3	14.11	7	4.70	11.21
Roof beams	3	14.40	5	10.50	13.69

Table 3 No of destructive tests for beams and columns at each floor and average values for tensile yielding and rupture steel strength

Members	N° of destructive tests	Yield Average (MPa)	Rupture Average (MPa)
Foundation columns	2	324.7	474.8
Foundation beams	2	315.7	448.9
Ground floor columns	3	333.1	471.2
Ground floor beams	2	338.1	523.9
First floor columns	-	-	-
First floor beams	2	321.2	476.4
Roof columns	-	-	-
Roof beams	2	320.5	478.6

There are also some methods to account for the uncertainties in the material properties definition. It is possible for example to consider material properties distributions based on their typical values for the geographical region of interest and to update these distributions based on the in situ tests (Jalayer *et al*. 2011, Miano *et al*. 2017).

Table 3 summarizes the values used at each floor for both columns and beams for the tensile yielding and rupture steel strength. With respect to first floor and roof columns, the values corresponding to ground floor have been used for the tensile yielding and rupture steel strength, due to impossibility to have in situ tests.

The cost of the tests is based on different contributions, mainly related to concrete carrots extraction, concrete carrots test, steel bars extraction, steel bars test and ultrasonic tests. The costs proposed herein are related to Campania region (Regional price list for Campania, 2018). Table 4 shows the costs for the different operations related to in situ tests, both for concrete and steel.

Table 4 Costs for the different operations related to in situ tests

Operation	Cost per unit (euro)	Units	Total cost (euro)
Concrete carrot extraction	58,20	40	2328
Concrete carrot test	171,38	40	6855
Steel bar extraction	78,52	13	1021
Steel bar test	22	13	286
Ultrasonic test	50	62	3100

The total sum of the in situ tests program is 13590 euro. Each specific design has its own difficulties in sampling the structure, so in each case some type of tests can be very easy, while some others can be very difficult. So, it's obvious that the real applications are quite different with respect to parametric calculations, but the comparison can be seen as strong tendency line and guideline for application. However, in order to theoretically minimize the total cost in order to achieve KL2, the maximum amount of non-destructive tests should be planned. These tests are considered possible, assuming ideally that all the structure is available for testing. Otherwise, as shown for the case study, the tests should be planned base on the specific condition of the case study.

2.3 Pushover analysis results and vulnerability index (ζ_E) calculation

Pushover analysis has been carried out for the case study by following the Italian seismic code (NTC 2008), based on the properties calculated with respect to KL2. A number of 16 pushovers has been done, using two different distributions of forces in the two horizontal directions and using a possible eccentricity of the 5% (called *ecc* in Figure 4 and measured perpendicularly to the direction of the seismic action) with respect to the geometrical barycentre of the building. The two distributions forces are suggested from Italian code (NTC 2008, Commentary 2009). The first distribution (*D1* in Figure 4) corresponds to an acceleration pattern proportional to the shape of the fundamental vibration mode in the considered direction of the building. The second distribution (*D2* in Figure 4), instead, is obtained from an uniform acceleration pattern along the height of the building. Figure 4 shows the pushover curves in terms of base shear versus top displacement. The sixteen pushovers are differentiated by colours in group of four, where each group represent a certain direction of application of the forces (the reference axes are shown in Figure 2).

From the pushover backbones, it is possible to see that the achieved displacements are not very high. They correspond to an interstory drift close to 1%. It clearly means that the structural system is not able to express significant plastic excursions. At the same time, the structural system is also not able to express an high stiffness/strength.

For each of the limit state required for a school type building, e.g., Operability (*OP*), Damage Limitation (*DL*)

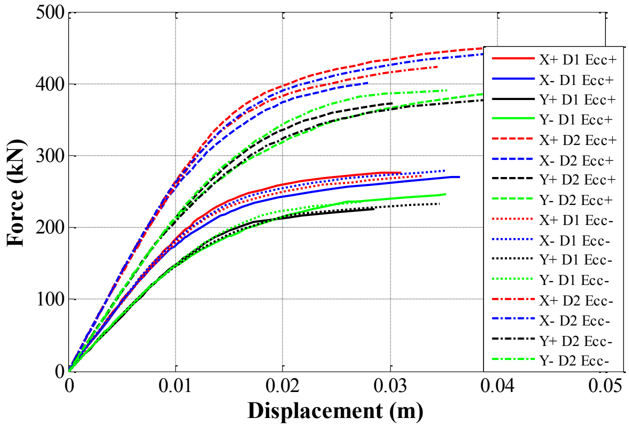

Fig. 4 Pushover curves for the case study before retrofit operations

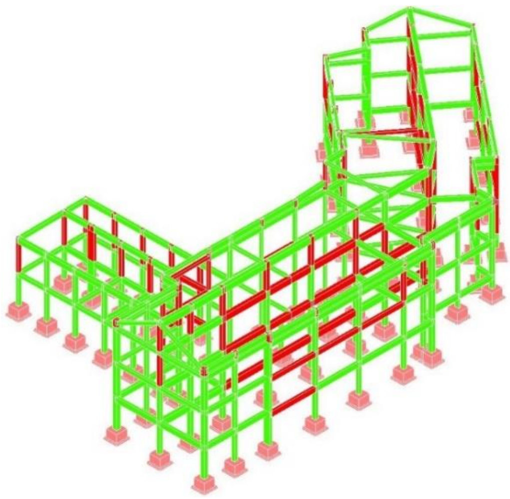

Fig. 5 Safety verifications for the case study before retrofit operations

and Life Safety (*LS*) limit states, the members verifications have been done, showing a situation of strong deficiency. It is interesting to show the safety verifications for the *LS* limit state. In particular, Figure 5 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied, showing that a high number of columns and beams result unsafe.

The safety verification have been implemented also per the other limit states (e.g., *OP* and *DL* limit states), showing a lot of deficiencies in the structural members. However, to measure the level of seismic safety after retrofit, the Italian code (NTC 2008 and NTC 2018) suggests to verify the vulnerability index with reference to *LS* limit state. For this reason, the bare building and its retrofitted versions are compared based on the vulnerability index calculated with respect to *LS* limit state, verifying a part in addition the condition of the members in the *OP* and *DL* limit states.

Definitively, in order to define if the building is safe with respect to seismic actions, the software (CDS 2018) calculates the vulnerability index (ζ_E), also called in literature seismic safety factor (Frascadore *et al*. 2015), that is a very useful parameter to measure the vulnerability of the structure. The pushover curve is an essential tool for the application of the capacity spectrum method (CSM; Vidic *et al*. 1994) that allows for the determination of the building response for earthquakes of a given spectral shape. All the steps of the procedure for calculate the SRI are well described in Frascadore *et al*. 2015. The procedure starts from the pushover response in terms of the Multi Degree of Freedom (MDOF) system and passes to the response in terms of the corresponding Singol Degree of Freedom (SDOF). The parameters that characterize the SDOF, period T^*, yield strength F^*_y, and ultimate displacement d^*_u, allow to calculate the return period capacity, and therefore the peak ground acceleration capacity, for which the crisis mechanism is reached. The procedure for the quantification of the ζ_E is implemented in the ADRS space (Acceleration Displacement Response Spectrum, Fajfar 1999, Fajfar 2000), in which the abscissas are the spectral displacements and the ordinates are the spectral accelerations. It consists in scaling the elastic spectrum of seismic demand, for small decrements of the return period T_R, until the spectrum that contains the point performance (Sae; Sde) of the equivalent SDOF is found, identified by the line of inclination T^* and the displacement d^*_{maxSLV}.

Finally, ζ_E is defined as the ratio between the demand peak ground acceleration (PGA), based on the seismic actions prescribed from the code for the *LS* limit state, and the capacity PGA of the building:

$$\zeta_E = \frac{PGA_{LS\_Capacity}}{PGA_{LS\_Demand}} \tag{1}$$

where $PGA_{LS\_Capacity}$ is the PGA corresponding to the achievement of the first crisis related to *LS* limit state inside the building, while PGA_{LS_Demand} is the PGA obtained from the elastic code spectrum for the specific site with reference to the *LS* limit state. It can be noted that this ratio between these two accelerations is directly related to the measurement of the seismic vulnerability of the structure with reference to the achievement of the crisis condition for the *LS* limit state. There are 16 SDOF systems associated to the 16 pushovers. The minimum value among the 16 values of the ζ_E related to the 16 SDOF systems is considered as ζ_E of the structure.

The final value of the ζ_E is 0.26. Thus, the structure is not safe in terms of seismic actions. Moreover, also the verifications in terms of static actions are not satisfied.

3. Existing building seismic retrofit

The assessment process for the case study shows that retrofit operations are needed for the case study. Moreover, this is confirmed by the total absence of the most important structural details to prevent brittle failure mechanisms in the bare building. For example, there is a quite total absence of stirrups in the joints. In some cases, the continuity between the two columns is interrupted by the presence of the slab. Finally, also the requirements given by the new codes related to length of lap-splices and anchorage are not respected.

In general, the main goal of the retrofit design is to prevent premature non-ductile failure modes and to increase their ductility and strength. In addition, the lateral displacements need to be limited and as uniform as possible over the height of the structure to prevent soft story mechanism. To control lateral drift by keeping them below the target displacement, one effective strategy for concrete moment frame is to add lateral stiffness, e.g., by adding a shear wall, to reduce the period and decrease the resulting displacements. In some cases, to avoid the restriction of use of building for a long time, methods based on a quick application, such as fiber reinforced polymers (FRP), can be useful. The application of this method increases the global deformation capacity of the structure and then its dissipating global performance, due to the confinement effect on the existing concrete and allows to fully exploit such capacity by avoiding brittle collapses modes. However, there are many practical retrofit options (Moehle 2000, Thermou and Elnashai 2005, Di Ludovico *et al*. 2008, Calvi 2013, Valarinho *et al*. 2013, Guneyisi and Azez 2016, Saribiyik and Caglar 2016, Formisano *et al*. 2017, Cabral-Fonseca *et al*. 2018, Miano *et al*. 2018a).

Moreover, the study is carried out with reference to different values of ζ_E, calculated for the *LS* limit state as prescribed from the code. In particular, three thresholds of improvement of the seismic performed are identified (e.g. values 0.60, 0.80 and 1 of ζ_E, where the value 1 means that the structure is safe with respect to seismic actions). It's important to highlight that coherently with the current Italian code, in case of seismic retrofit of a school building, the minimum value of the ζ_E after the retrofit operations should be 0.60.

3.1 Retrofit strategies

Different retrofit options are considered: 1) all columns and beams of the building are steel jacketed, 2) all columns and beams of the building are FRP wrapped, 3) all columns of the frame are RC jacketed and all the beams are FRP wrapped and 4) all columns of the frame are RC jacketed and all the beams are steel jacketed.

It is to note that load tests were done on the slabs. The slabs showed a safe condition both based on strength and deformation verifications. Then, it was important to avoid too invasive modifications on the slab configuration through the retrofit strategies. Due to this reason, it has been considered not convenient to have RC jacketing of the beams. Instead, mainly the columns, but also the beams, showed evidence of structural malfunction. This was due to the poor quality of the concrete and to the absence of adequate seismic-detailing requirements, as shown in Figure 6.

In addition to the different retrofit options listed before, there are some main interventions needed for all the options: 1) in the foundations, a 60 cm thick slab has been realized; this type of operation allows to have a better distributions of the actions on the ground; 2) some interventions on the roof have been realized in order to replace some old wood beams; 3) in the foundation floor, the addition of reinforced concrete shear walls along the external perimeter of the

Fig. 6 a) Low quality concrete in a perimeter column; b) Total absence of stirrups in a corner column

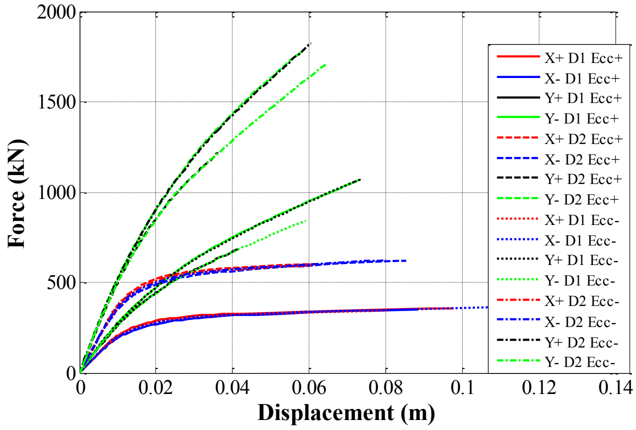

Fig. 7 Pushover curves for the case study after steel jacketing

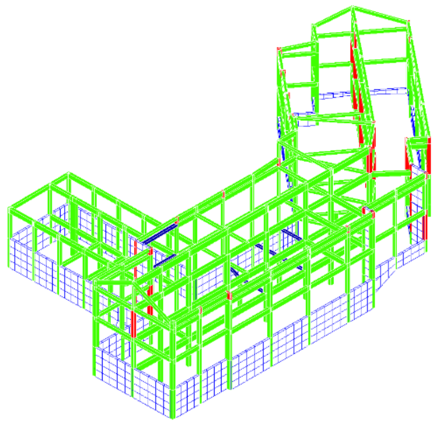

Fig. 8 Safety verifications for the case study after steel jacketing

building has been realized; 4) some new reinforced concrete beams, that were not present in the existing building, have been added to guarantee some crucial connections in both directions between all the structural members.

It is to note that the effect on the structural behaviour due to the realization of the concrete shear walls in the foundation floor is relevant. In fact, these walls carry the actions from the surrounding ground and distribute the loads linearly on the foundation slab. Moreover, they incorporated some short columns, preventing premature brittle failures of these columns. Anyway, the failure modes comparison among the different retrofit options is discussed in Section 3.1.5.

The different retrofit options have been compared both in terms of structural performance and in terms of structural costs. If possible, the same retrofit option has been designed more times in order to achieve different levels of ζ_E (e.g. 0.6, 0.8 and 1.0). Obviously, for some retrofit options, it is impossible to achieve all the different levels of ζ_E, due to technological limitations (e.g., there is a maximum number of FRP sheets that can be applied).

3.1.1 Columns and the beams steel jacketing

In this retrofit option, all columns and beams have been steel jacketed. The steel jacketing has been performed through steel plates wrapped completely around the beams plus angular plates in the corners along members length. As for the bare building, pushover analysis has been carried out by following the Italian seismic code (NTC 08) for the case study retrofitted building. Figure 7 shows the pushover curves in terms of base shear versus top displacement, using the same reference system presented in Figure 2. In the legend, *D* indicates the distribution and *ecc* the eccentricity, according with the definitions provided in Section 2.3.

As one may note from the pushover curves, the application of steel jacketing to columns and beams allows reaching a more ductile global behaviour with respect to the bare building. However, the very poor quality of the existing concrete does not allow reaching a significant increase of strength for all the curves.

Figure 8 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied, showing that 32 columns result unsafe. The members shown in blue are new reinforced concrete beams (in the existing building, they were not present) to guarantee some crucial connections in both directions between all the structural members. In addition, in the foundation floor, the reinforced concrete walls are showed in Figure 7.

After achieving a ζ_E value of about 0.45, although the thickness and the typology of steel are increased, the value of ζ_E remains more or less the same. Then, this retrofit option doesn't allow to reach the value of ζ_E of 0.6 and obviously also the values of 0.8 and 1.0.

3.1.2 Columns and the beams FRP wrapping

In this retrofit option, all columns and beams of the building have been wrapped using carbon FRP sheets of different thickness and a maximum number of sheets equal to 3. Figure 9 shows the pushover curves in terms of base shear versus top displacement, using the same reference system presented in Figure 2. In the legend, *D* indicates the distribution and *ecc* the eccentricity, according with the definitions provided in Section 2.3.

As one may note from the pushover curves, the application of FRP wrapping to columns and beams allows reaching a more ductile global behaviour (as for the steel

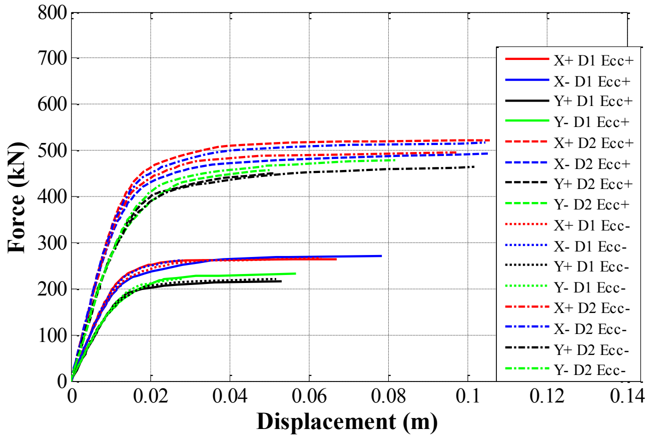

Fig. 9 Pushover curves for the case study after FRP wrapping

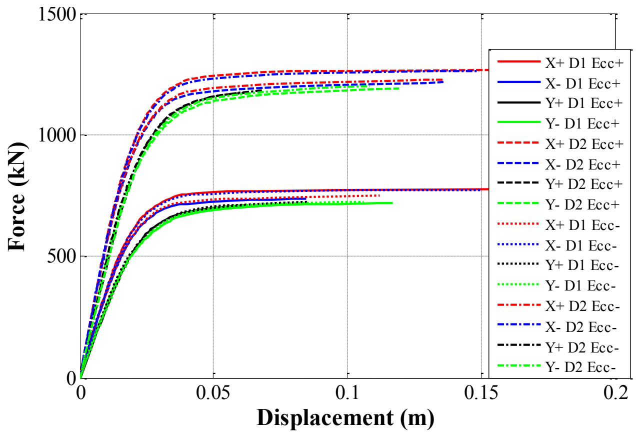

Fig. 11 Pushover curves for the case study after RC jacketing of the columns and FRP wrapping of the beams

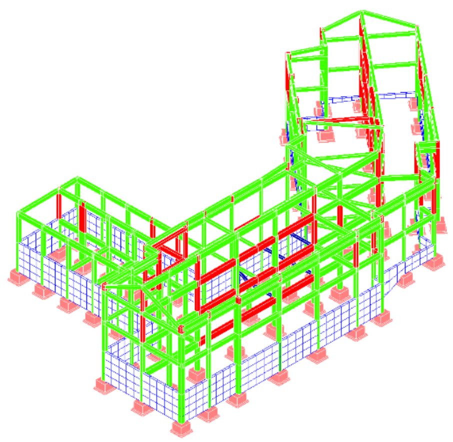

Fig. 10 Safety verifications for the case study after FRP wrapping

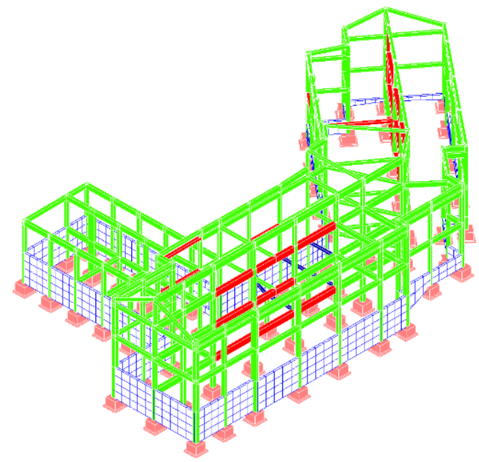

Fig. 12 Safety verifications for the case study after RC jacketing of the columns and FRP wrapping of the beams

jacketing). However, also for this option, the very poor quality of the existing concrete does not allow reaching a significant increase of strength for all the curves.

Figure 10 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied, showing that 44 columns and 26 beams result unsafe.

After achieving a ζ_E value of about 0.3, although the thickness and the typology of FRP wrapping are increased, the value of ζ_E remains more or less the same. Therefore, this retrofit option doesn't allow to reach the value of ζ_E of 0.6 and obviously also the values of 0.8 and 1.0.

3.1.3 Columns RC jacketing and beams FRP wrapping

In this retrofit option, all the columns of the building have been RC jacketed, with different thicknesses of the jacket depending on the specific condition of the columns, while all the beams have been wrapped using carbon FRP of different thickness and a maximum number of sheets equal to 3. Figure 11 shows the pushover curves in terms of base shear versus top displacement, using the same reference system presented in Figure 2. In the legend, *D* indicates the distribution and *ecc* the eccentricity, according with the definitions provided in Section 2.3.

As one may note from the pushover curves, this option gives better results with respect to the precedent ones. In particular, the RC jacketing of the columns allows reaching a significant increase of strength, while also the FRP wrapping of the beams contributes to give a larger ductility to the structural system. However, these beams still show mainly flexural failures, due to the very poor quality of the concrete. It is to note that a more in depth study on the typology and thickness of the fibers (that was out of the goals for the case study design project) could help to reach better results in increasing both strength and ductility.

Figure 12 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied, showing that 2 columns and 33 beams result unsafe.

After achieving a ζ_E value of about 0.60, although the thickness and the typology of FRP wrapping for the beams are increased, the value of the ζ_E remains more or less the same. Therefore, this retrofit option doesn't allow to reach the value of ζ_E of 0.8 and obviously also the value of 1.0.

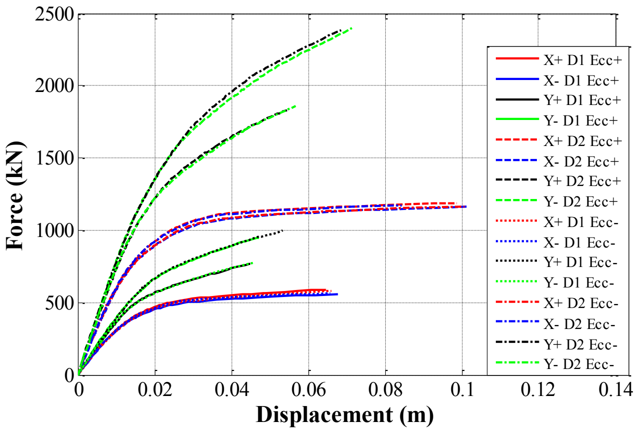

Fig. 13 Pushover curves for the case study after RC jacketing of the columns and steel jacketing of the beams (ζ_E =0.6)

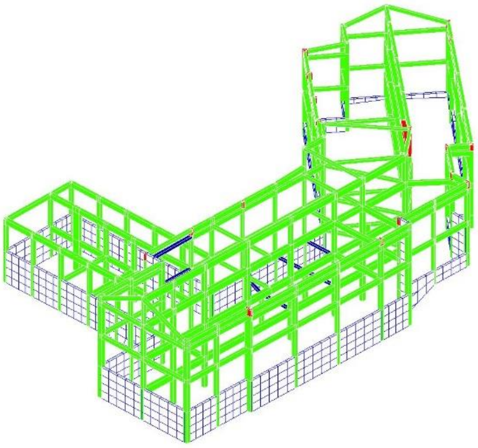
Fig. 14 Safety verifications for the case study after RC jacketing of the columns and steel jacketing of the beams (ζ_E =0.6)

3.1.4 Columns RC jacketing and beams steel jacketing

In this retrofit option, all the columns of the building have been RC jacketed, with different thicknesses of the jacket depending on the specific condition of the columns, while all the beams have been steel jacketed. The steel jacketing has been realized through steel plates wrapped completely around the beams plus angular plates in the corners along members length. Figure 13 shows the pushover curves in terms of base shear versus top displacement for achieving a ζ_E value equal to 0.6, using the same reference system presented in Figure 2. In the legend, *D* indicates the distribution and *ecc* the eccentricity, according with the definitions provided in Section 2.3.

Figure 14 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied, showing that 19 columns and 1 beam result unsafe.

Figure 15 shows the pushover curves in terms of base shear versus top displacement for achieving a ζ_E value equal to 0.8.

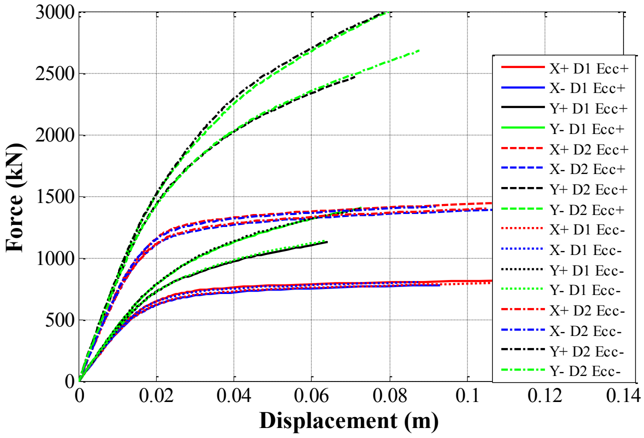

Fig. 15 Pushover curves for the case study after RC jacketing of the columns and steel jacketing of the beams (ζ_E =0.8)

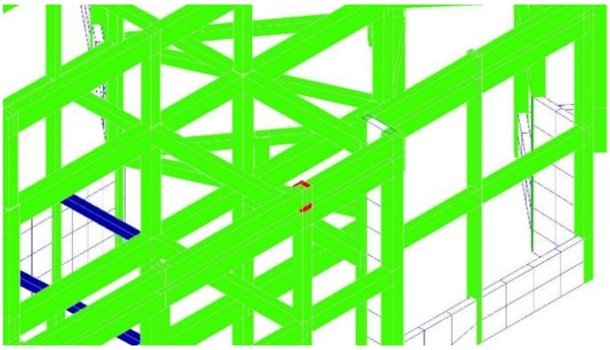
Fig. 16 Safety verifications for the case study after RC jacketing of the columns and steel jacketing of the beams (ζ_E=0.8)

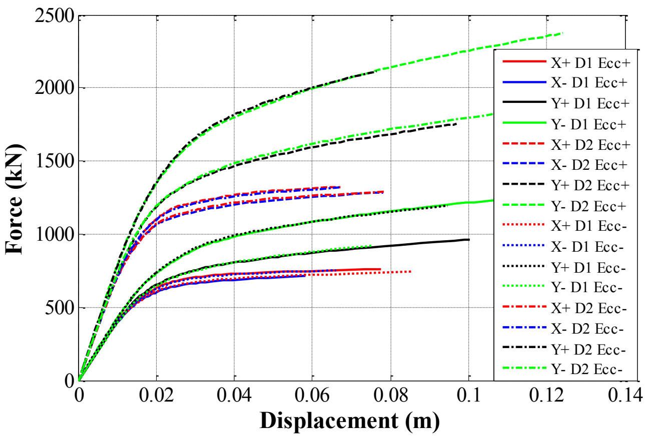

Fig. 17 Pushover curves for the case study after RC jacketing of the columns and steel jacketing of the beams (ζ_E=1.0)

Figure 16 shows for the *LS* limit state with red colour the members for which the safety verifications are not satisfied and with green colour the members for which the safety verifications are satisfied. In particular, in this case just one column results unsafe, as shown in the zoom provided in Figure 16.

Figure 17 shows the pushover curves in terms of base shear versus top displacement for achieving a ζ_E value equal to 1.0.

Post-intervention pushover curves presented in Figure 17 show a general improvement in the seismic response for the whole performance of the building Specifically, the curves show the increase in deformation and resistance capacity of the structure as a result of structural project interventions. The verification of the members for the different limit states have been done, showing a global condition of seismic safety. The results show also the capacity of the reinforced and unreinforced structural elements to resist the static actions due to the gravitational loads calculated based on the recommendations provided in the code (NTC 2008 and NTC 2018). Definitively, in this case, all the members result safe from the verifications.

3.1.5 Comparisons

The retrofit options are compared in terms of the number of failure modes (e.g. shear or flexural failure modes) in Table 5. It is note that each retrofit option shows a trend of the failure modes. The full steel jacketing option shows a clear weakness in the safety verifications of the columns. Also the full FRP wrapping shows problems in columns both with shear and with flexural failure modes. This allows to conclude that the RC jacketing of the columns is crucial to achieve the seismic safety. This result could be considered a result of two main issues: 1) the very poor concrete of the existing columns; 2) the few steel bars used in the existing columns. The only way to solve these problems seemed to be increasing the concrete cross-section in the columns and increasing the number of steel bars in the same columns. With reference to beams, instead, the only solution has been the steel jacketing. However, as said before, some new reinforced concrete beams, that were not present in the existing building, have been added to guarantee some crucial connections in both directions between all the structural members.

Table 5 Failure modes comparison among the different retrofit options

Retrofit option	Failure mode	Columns	Beams
Steel jacketing (ζ_E =0.45)	Shear	28	0
	Flexure	4	0
FRP wrapping (ζ_E =0.30)	Shear	6	13
	Flexure	38	13
RC jacketing for columns and FRP wrapping for beams (ζ_E =0.60)	Shear	0	10
	Flexure	2	23
RC jacketing for columns and steel jacketing for beams (ζ_E =0.60)	Shear	3	1
	Flexure	16	0
RC jacketing for columns and steel jacketing for beams (ζ_E =0.80)	Shear	0	0
	Flexure	1	0
RC jacketing for columns and steel jacketing for beams (ζ_E =1.00)	Shear	0	0
	Flexure	0	0

In addition, it is important to give an insight about the importance of the perimeter walls in the foundation floor. If the walls are removed from the retrofit option of RC jacketing for columns and steel jacketing for beams (ζ_E=1), ζ_E becomes 0.30 and there are 5 beams that fail in shear and 17 beams that fail in flexure. It is important to note as the wall have two main contribution, e.g. giving more stiffness to the overall structure (in particular in the weak direction) and allowing to the beams to unload on them.

4. Structural costs assessment and comparison among the different retrofit operations

The paragraph summarizes the costs related to structure in elevation of the buildings (excluding roof and foundations retrofit operations). With respect to the different retrofit options, the main contributions to the total retrofit cost for the structure are related to:

- ➢ RC jacketing: the manufacturing works include mainly in depth removal of the damaged concrete, surface cleaning of the concrete, passivating material to upgrade the condition of the steel bars, formworks for the concrete, new concrete and steel bars for the jacketing, perforations for the steel bars through the floors and mortar injections to close the holes, transport to landfill of the wreckage material plus some minor contributions.
- ➢ Steel jacketing: the manufacturing works include mainly in depth removal of the damaged concrete, surface cleaning of the concrete, passivating material to upgrade the condition of the steel bars, new steel plates for the jacketing, anchor bolts to solidarize steel plates with the RC beam, transport to landfill of the wreckage material plus some minor contributions.
- ➢ FRP wrapping: the manufacturing works include mainly crack injections, sand blasting, primer, putty, saturant, and demolition, reconstruction and painting of partition walls.

Moreover, for each of the retrofit options, the walls cost has been considering. In particular, the manufacturing works include mainly preparation of the surface for the concrete flow, formworks for the concrete, new concrete and steel bars for the wall, perforations for the steel bars through the floors and mortar injections to close the holes, transport to landfill of the wreckage material plus some minor contributions. The cost of the walls is 1053 euro for cubic meter.

The unit costs related to these operations should be multiplied for the quantities of each operation in order to have an overall estimation of the retrofit costs. Definitively the costs for square meter are: 1) RC jacketing: 4112 euro for cubic meter; 2) Steel jacketing: 3808 euro for cubic meter; 3) FRP wrapping: 270 euro for square meter.

Table 6 shows the retrofit options, ζ_E after retrofit and the total retrofit operations cost.

The real retrofit design project has assumed as final solution the RC jacketing for columns and steel jacketing for beams with a ζ_E equal to 1. In fact, considering that Italian code requires for the school building at least a ζ_E equal to 0.60 and considering the small cost difference between the four options that achieve 0.60, the final choice went toward the complete seismic safety.

Table 6 Retrofit options, ζ_E after retrofit and total retrofit operations cost

Retrofit option	ζ_E after retrofit operations	Retrofit operations cost including in situ tests (euro)
Steel jacketing	0.45	563035
FRP wrapping	0.30	358490
RC jacketing for columns and FRP wrapping for beams	0.60	467580
RC jacketing for columns and steel jacketing for beams	0.60	429700
RC jacketing for columns and steel jacketing for beams	0.80	460810
RC jacketing for columns and steel jacketing for beams	1.00	482070

5. Conclusions

The retrofit of existing structures in highly seismic zones is one of the most challenging issue in earthquake engineering. A school building located in Avellino province (Italy) is used herein as case study to show a real retrofit design case study. The 3D model of the building has been produced using the commercial software for structural calculation CDS 2018. The analysis of seismic vulnerability carried out on the mentioned building has highlighted deficiencies in both static and seismic load conditions. Retrofit operations have been implemented in order to achieve the seismic safety condition for the real case study school building. In particular, different retrofit options are considered: 1) all columns and beams of the building are steel jacketed, 2) all columns and beams of the building are FRP wrapped, 3) all columns of the frame are RC jacketed and all the beams are FRP wrapped and 4) all columns of the frame are RC jacketed and all the beams are steel jacketed. If possible, the same retrofit option has been designed more times in order to achieve different levels of ζ_E (0.6 and 0.8 of the seismic safety and the complete seismic safety).

The retrofit structural costs related to structure in elevation of the buildings (excluding roof and foundations retrofit operations) have been calculated for each case and have been summed up to the costs of the in situ in tests for achieve the desired knowledge level in order to have a full cost of the retrofit design project.

Finally, the best solution for the case study building is chosen based on the results in terms of structural performance and cost among the different retrofit options.

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