Seismic Upgrade of Beam-Column Joints with FRP Reinforcement

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Abstract

Many opportunities are becoming available for using composite materials to strengthen/upgrade existing reinforced concrete (RC) structures. This paper focuses on a new technique for the seismic upgrade of RC beam-column connections in gravity load-designed (GLD) frames by the application (combined or not) of FRP rods and laminates. The FRP rods provide flexural strengthening, whereas the lay-up laminates provide confinement and shear strengthening. Along with the modeling of such upgraded connections to assess the increase of strength and/or ductility provided by the composite reinforcement, an experimental program was planned and it is being undertaken. A preview of it is given in this paper together with an explanation of its philosophy; furthermore, interesting preliminary results are presented and discussed. It appears that the proposed upgrade method will have a significant impact of the engineering practice in the near future.

Introduction

The upgrade of the seismic performances of existing reinforced concrete (RC), gravity load designed (GLD) structures is an important issue that involves economic and social aspects in different areas of the world like, for example, Europe, USA and Japan. In fact, the RC frames, designed without seismic provisions, are often characterized by an unsatisfactory structural behavior due to the low available ductility and the lack of a strength hierarchy inducing global failure mechanisms. In these cases, some constructive details of the GLD frames can be pointed out as the potential critical causes of brittle failure mechanisms, which are sensitive to the cyclic damage. For example in a column, the lack of appropriate size and spacing of ties, which does not guarantee the required level of confinement, can cause the collapse of the column end, resulting in crushing of the not confined concrete, instability of the steel reinforcing bars in compression and pull out of those in tension. A critical region in an RC frame is the beam-column connection, where different constructive details can originate local failures, such as the shear collapse of the panel due to the lack of transverse reinforcement. Moreover, either for the scarce stirrup reinforcement or for the extreme loads due to a seismic event a beam can show a shear failure. In general, the lack of a design based on the strength hierarchy influences the global behavior of GLD frames and is responsible for the low dissipative capacity [Cosenza and Manfredi 1997, White and Mosalam 1997]. The present paper focuses on the problems connected with the seismic upgrading of GLD frames and, in particular, on beam-column joints that can be knee, interior, exterior, or corner joints depending on their vertical and horizontal position in the structure. On interior and exterior GLD connections (a large part of the joints of a building), an experimental study was performed by
Beres et al. [1996]. They underlined seven critical details that could be potential causes of failure in a non-seismically designed structure subject to an earthquake. Four of the seven were related to the columns, they are:

- Longitudinal reinforcement less than 2% of the concrete cross section
- Lapped splices of the longitudinal reinforcement above the construction joint
- Scarce confinement provided by the ties, and
- Construction joints above and below the beam-column connection

The remaining three are related to:

- Discontinuous positive reinforcement in the beam
- Lack of transverse reinforcement in the panel, and
- Weak column-strong beam condition.

Composite materials, known as fiber reinforced polymers (FRP), have shown a great potential for the strengthening of reinforced concrete structures [Dolan et al. 1999] in the forms of:

- Lay-up laminates externally bonded to a concrete member to increase flexural and shear capacity as well as to provide concrete confinement [Alkhrdhaji et al. 1999], and
- Near surface mounted (NSM) reinforcement to increase flexural and shear strength [De Lorenzis et al. 2000].

At present, studies on the possible application of composite materials for the seismic upgrade of RC beam-column connections are at early stage [Pantelides et al. 2000]. The present research focuses on this topic with the objective of evaluating the opportunities offered by using FRP sheets and NSM rods in strengthening GLD joints. In order to assess the validity of this strengthening methodology, a detailed experimental program is proposed. The program will analyze the effects of FRP reinforcement detailing and the other parameters on the global behavior of the beam-column connection and its failure mechanisms. The test results and their analysis will be used to compile design and construction guidelines on the seismic retrofitting and upgrade of RC beam-column joints with FRP sheets and NSM rods. This paper will provide a first insight on the research program previously described. The outcomes of the first two tests are presented and the encouraging results are discussed.

**Research Objectives**

The goal of this research is to investigate the effects of the FRP reinforcement detailing on the behavior of the beam-column connection, its failure mechanisms, and its ductility. In particular, the planned experimental program is expected to demonstrate that, by means of targeted strengthening, it is possible to establish a strength hierarchy in the member. This hierarchy starts from lowest level (i.e., column failure) given by the virgin joint. The first level of FRP upgrade moves the failure to the panel, and then, further strengthening moves it to the beam. The influence of the axial load applied to the column is also investigated. The final results will be used to develop design guidelines that, for different situations, would allow the choice of the

upgrade level for the joint depending on the desired strength and/or ductility. Ultimately, in same cases, the engineer could modify the performance of an existing structure with an economical and sound technology.

Test specimens

In choosing the dimensions of the specimen, typical frame and geometrical ratios were taken in account even though some scaling was adopted to maintain the specimen size and weight to a manageable level. The specimens cross sections and the steel reinforcement used are shown in Figure 1.

The specimen design was carried out following the recommendations of a building code [ACI 318-63] pre-dating the current one, as it is representative of a large number of existing buildings. These code provisions were used to determine the spacing of steel stirrups and ties. The amount of longitudinal steel reinforcement for the column was fixed equal to about 2% of its concrete gross cross-sectional area (ACI 318-63 allows a minimum of 1%). The concrete cover was equal to 38 mm (1.5 in). Concerning the materials, the concrete had compressive strength \( f'_{c} = 31.7 \) MPa (4530 psi) and the steel was characterized by \( f_{y} = 420 \) MPa (60000 psi).

For flexural strengthening of the column, #3 (i.e., 9.5 mm= 0.375 in diam.) CFRP rods were used, having the modulus of elasticity \( E_{fr} = 106,400 \) MPa (15200 ksi) and the ultimate tensile strength \( f_{ru}^{*} = 1,904 \) MPa (272 ksi). The confinement by lay-up jacketing was accomplished with two plies of CFRP. Each ply had a fiber-only thickness of 0.165 mm (0.0065 in) characterized by a modulus of elasticity \( E_{fs} = 229,600 \) MPa (32800 ksi) and the ultimate tensile strength \( f_{su}^{*} = 3,465 \) MPa (495 ksi). The FRP wrapping was extended for 38 cm (15 in) over the length of the column and an overlap of 10 cm (4 in) was realized for each ply.

The set-up of each specimen on the laboratory floor is shown in Figure 2. At one of the column ends a constant axial load \( P \) is applied by means of an hydraulic jack independently operated. At the other end of the column, a load cell is placed to record the applied load \( P \). The restraints at the extremes of the columns simulate hinges that allow rotation by means of a pair of steel bars on both sides of the element. Two additional shear loads are applied on the extremes of each beam. Even in this case the jacks are independently operated. A load cell on each cylinder contacting the beam records the applied force. A plywood sheet between the specimen and the floor limits the friction and allows for the free movement of the beams and column.

For the specimen where the column is to be strengthened with NSM rods, eight wooden strips were nailed to the forms so that grooving the column and drilling through the panel for the subsequent installation of the rods could be performed with less difficulty and greater precision. Figure 3 shows this detail. It has been demonstrated in other projects that the grooving and drilling is possible in the field with conventional tools. Figure 4 shows the case of NSM rods mounted in a bridge column and anchored in the footing with a length of 0.5 m (18 in) [Alkhrdaji and Nanni 2000].

Experimental program

The first phase of the experimental program consists of 12 tests. In particular, two series of tests are planned. Each series is characterized by the same axial load applied to the column. The difference among the six specimens relates to the level of upgrading, designed to verify expected steps of the strength hierarchy. The first specimen is the “virgin” joint (1.a or 2.a), like those
designed and built without seismic provisions. The other five are upgraded with FRP reinforcement in the following ways: two of them show respectively wrapping (1.b or 2.b) and wrapping combined with NSM rods in the column (1.c or 2.c). In the following two, both column and panel are strengthened, the latter with sheets (1.d or 2.d) and sheets and rods (1.e and 2.e), respectively. In the last one, the wrapping of the beams is added to the strengthening of columns and panel (1.f or 2.f). This program is summarized in the in Table 1 (for the symbols see Figure 1 and Appendix A). Figure 5 shows the difference between specimen types a or b and specimen types c, d, e, and f.

The influence of the compressive stress acting the column on the resistant mechanism of the connection will be assessed by comparing specimens belonging to the series. Axial loads reported in Table 1 induce in the columns an average compressive stress equal to 6 MPa (860 psi) for series 1 and 3 MPa (430 psi) for series 2. The larger value of this stress takes into account the increase of axial load due to a seismic event, while the smaller refers to serviceability conditions.

### Loading arrangement

Once the specimen is in place (as depicted in Figure 3), the loading process includes the following steps:

- **Axial column load application**: this is the preliminary step, during which gradually the prefixed load P is applied to the column. During the application of the axial load, the beam ends are free in order to allow the small displacements in their transverse direction.

- **Gravity beam loads application**: after the axial load has reached the fixed value and is kept constant, the beam load setup is arranged. Two equal shear forces are then applied to the beam ends in order to simulate the effect of gravity loads. The amount of the total load was calculated to reproduce on the beams the serviceability situation. In particular, the adopted value V= 40.1 kN (9 kips) generates a maximum flexural moment equal to half of the design moment obtained according to ACI provisions. The beam-column joint configuration in the laboratory during this phase is shown in the Figure 6.

- **Reversed load cycles**: with this step the earthquake simulation starts. As soon as the beam shear is unequal at each side of the connection, a flexural moment and shear forces are generated in the column to maintain equilibrium. The increment/decrement ?V= 4.45 kN (1 kip) was established so that at least six reversed load cycles were needed before reaching the design moment strength in the column calculated according to ACI provisions. The algebraic sum of beams shears and compressive column axial force is kept constant during the test. Figure 7a schematically shows the shears application performed by maintaining the jacks from the same side of the beams and increasing/decreasing their forces. For every shear force increment/decrement, three repetitions are performed. Once zero shear force is reached at one of the beam ends (i.e., nine load cycles), the load configuration is changed switching the cylinders on the beam ends as shown in Figure 7b.

The three steps described above are shown in Figure 8 which represents the temporal load application recorded during a test.
Preliminary experimental results

After the first two tests (1a and 1c in Table 1) some significant results were obtained. First, a comparison between virgin and strengthened joint underlines an significant increase of the column strength; a change also in the column stiffness can be observed. Moreover, while the former collapsed by compressive failure at the critical cross sections in the column, in the latter, the crisis migrated to the panel.

In the strengthened specimen, panel failure occurred without using the total column capacity, but, in spite of this condition, the ultimate column shear was about 1.6 times that of the virgin column. These results can be clearly observed in Figure 9.

Additional information was also given by the strain gauges applied to the CFRP laminate in the direction of the fibers (i.e., hoop direction) on two lines parallel to the axis of the beams and placed, respectively, at 5 cm (2 in) (1 and 13 in Figure 10) and 10 cm (4 in) (2 and 14 in Figure 10) from the end of the column. The strains recorded on the horizontal superior face of the column (1 and 2) and on its vertical side (13 and 14) have substantially similar values for tensile stress on the latter. However, during the cyclic load, when the column face is in compression, the recorded strains increase due to the increasing demanded degree of confinement. Moreover, the high strains of the FRP wrapping (about 0.0023 in/in), corresponding to the failure of the panel, underline the effectiveness of its confinement function. Finally, a comparison between the strains recorded by 13 and 14 allows to point out how the amount of demanded confinement decreases strongly when the distance from the nodal zone increases. The graphs reported in Figure 10 show all this.

Preliminary Analysis

Confirmation of the validity of the test set up and loading procedure was provided by a preliminary analysis of the results obtained for the virgin column. First, the moment-curvature relationship shows that the ultimate moment of the column 42.3 kNm (31.2 ft-kip) is practically identical to the experimental moment 42.5 kNm (31.3 ft-kip) calculated by equilibrium of the applied loads. This confirms that the test setup works as expected and the experimental behavior of the column was consistent with the theoretical prediction. As experimentally observed, the analysis confirms that the failure was due to concrete crushing, while at that instant the steel reinforcement on the tension side was yielded. For the strengthened specimen, a similar analysis is being carried out, adding the contribution of the additional FRP rods and the effect of the jacketing on its flexural strength.

Conclusions

The preliminary results of an experimental program confirmed that the combined action by FRP jacketing and near surface mounted FRP rods can be a promising and flexible retrofit technique for beam-column connections in GLD buildings. In fact, it could make it possible to modify strength and/or ductility of the joints by varying the combined use of FRP wrapping and NSM rods. In this way the design of the retrofitting could be oriented to induce a favourable collapse mode of the upgraded frame.
In order to accurately assess the influence of different key parameters (e.g., axial force, amount and type of FRP reinforcement), it is necessary to complete the experimental program described herein. The goal of the project is to define design criteria so that, known the initial conditions and loads, it will be possible to design the structural retrofit changing the failure mode and varying the improvement in terms of strength and/or ductility.

References

American Concrete Institute Committee, (1963) “Building Code Requirements for Reinforced Concrete” ACI 318-63, Detroit USA.


Acknowledgements

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Appendix A - Notations list

\( A_{col} \): area of longitudinal steel reinforcement on one side of the columns;
\( A_s \): area of superior longitudinal reinforcement of the beams;
\( A'_s \): area of inferior longitudinal reinforcement of the beams;
\( C \): dimension of each side of the columns and of minimum side of the beams;
\( f'_c \): compressive strength of concrete;
\( f_y \): yield stress of steel reinforcement;
\( D \): maximum dimension of the beams;
\( E_{fr} \): modulus of elasticity of CFRP rods;
\( E_{fs} \): modulus of elasticity of CFRP sheets;
\( P \): axial load on the columns;
\( s \): space between each stirrup in the columns;
\( s' \): space between each tie in the beams;
\( V_1 \): shear on beam 1 during reversed load cycles;
\( V_2 \): shear on beam 2 during reversed load cycles;
\( Vv \): shear on both beams due to vertical loads;

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<th>Spec.</th>
<th>Axial Load kips</th>
<th>C in.</th>
<th>$A_{col}$ bars</th>
<th>s in.</th>
<th>FRP sheets</th>
<th>FRP NSMr rods</th>
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<th>C in.</th>
<th>D in.</th>
<th>$A_s$ rods</th>
<th>$A_s'$ rods</th>
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Conversions:
1 kip = 4.448 kN
1 in. = 25.4 mm
#m = m/8 in.
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\[ V_1 = V_v \pm n \cdot V \]
\[ V_2 = V_v \pm n \cdot V \]

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Figure 8

Figure 9
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