Experimental testing on the structural capacity of coupling beams with non-anchored longitudinal rebars

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Abstract. Coupling beams are structural elements which connect shear walls to improve the lateral stiffness and allow to transfer shear forces, while possessing sufficient ductility to dissipate the energy produced due to the lateral displacement. An error in the construction of a 16-story buildings resulted in that only the bottom layer of the longitudinal reinforcement bars in all coupled beams are anchored into the lateral walls, the upper longitudinal bars are cut before anchored to the lateral walls. The reconstruction of all coupled beams in the building is technical and economically non-feasible therefore, to propose a more efficient and less costly retrofit procedure, an experimental study is undertaken to analyze the real structural capacity of these coupling beams. Carbon fiber reinforced polymer (CFRP) is chosen as the reinforcement material for the upper part of the beams. A total of three specimens are tested, two of them under cyclic quasi-static load to determine the flexural capacity of the cross section in the interface wall-coupling beam, and one of them under monotonic loading to determine the shear capacity of the cracked cross section. The results of the cyclic test showed a very limited contribution of the CFRP in the flexural capacity when the fiber is in tension. However, the flexural behavior of the section with the lower reinforcement in tension is not affected by the upper non-anchored bars and showed a flexural capacity and energy dissipation according to the existing rate of the steel reinforcement. The experimental results are used to develop a finite element model which reproduce the structural behavior of the beams with the construction error.

1 Introduction

Understanding the behavior of coupling beams is an important task of seismic design of buildings. These beams resist and transmit high shear forces and deformations, and they are important components in the energy dissipation during an seismic event.

An error in the construction of a 16-story building complex resulted in only the lower layer of longitudinal rebars for all coupling beams anchoring within the adjacent walls. The upper layers are sheared before entering the wall, as shown in Figure 1.

The reconstruction of all these beams is technically and economically unfeasible. Therefore, the objectives of the present study are to experimentally evaluate CFRP sheets as an retrofit alternative in order to restore the coupling beams performance, and to determine the structural capacity of coupling beams with continuity only in the lower layer of the longitudinal rebars.

There are no antecedents in the literature about the use of CRFP to replace longitudinal rebar and the use of numerical modeling to represent these crack pattern.

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Figure 1. [a] - [b] Testing specimens from coupling beam, [c] Reinforcement as-designed [d] Reinforcement as-built

2 Experimental program

2.1 Test specimen’s details

Three coupling beam-wall sub-assemblages are built, called CB-1, CB-2 and CB-3, taking as reference the story height, clear span between walls, cross sections dimensions and arrangement of the rebars as-built. These specimens represent the interaction between the coupling beam and the coupled walls.
The beam dimensions are 2.10m length and 0.25m x 1.10m cross section in the three specimens. The dimensions of the wall are 3.00m length and 0.25 x 0.60m cross-section in specimens CB-1, and CB-2 and 0.25 x 1.20m in CB-3. Figure 2 shows the geometry and rebar details of the specimens CB-1 and CB-2.

Post-tensioning lines are installed on the walls to simulate gravity loads. In specimens CB-1 and CB-2, two lines formed by a 0.6" diameter strand are placed, tensed up to 110kN, obtaining a total axial force equal to 220kN. In specimen CB-3, four lines formed by a 0.5" diameter strand are placed, tensed up to 95kN, obtaining a total axial force equal to 380kN.

2.2 Carbon fiber reinforced polymer

CB-1 is the control specimen which represents the as-built coupling beam. CB-2 is retrofitted with two layers of CFRP sheets (uniaxial and biaxial sheets) which are placed externally in only one face in the beam-wall interface.

2.3 Cyclic test protocol

The cyclic test consists of applying force to the free end of the beam in complete cycles controlled by displacement, using a servo-controlled static actuator with a maximum nominal capacity equal to 500kN. Figure 3 depicts cyclic displacement amplitude history.

Figure 2 [b] shows the test setup and the location of the LVDT (Linear Variation Differential Transformer) displacement transducers. The beam is mounted in a vertical position, with the wall ends restricted from sliding and turning with hydraulic jacks that apply constant pressure in the restrained degree of freedom direction.

The horizontal force application axis is at a height of 2.63m with respect to the test slab. The force transducer built into the structural actuator records the force applied in the test. The positive direction [+ ] of the force (push) produces tension at the extreme of the beam height where rebars anchor the wall and is associated with the positive branch [+ ] of the specimen response. The negative direction [− ] of the force (push) produces tension at the extreme of the beam height where rebars do not anchor the wall and is associated with the negative branch [− ] of the specimen response.

The transducer D1 is aligned with the actuator axis (force application axis) to record the lateral displacement of the beam top end with respect to a fixed reference system, and to perform displacement control.

3 Experimental results

3.1 Crack pattern

Figure 4 show the cracking evolution in the positive and negative branch in the CB-1 specimen test. In the positive branch distributed cracking is observed in the concrete in the beam-wall interface area. This cracking is produced by the bending of the beam in the presence of rebars in tension. In the negative branch a single crack occurred after exceeding the tensile strength of the concrete, due to the absence of rebars anchored in the wall.

In the final stage of the test, the cover concrete is spalled and the longitudinal steel bars of the beam are exposed. No signs of necking, buckling or shearing are observed in these bars.

3.2 Hysteretic behavior

Figure 5 shows the hysteretic behavior and the cyclic envelopes of specimens CB-1 and CB-2. The cyclic envelope is obtained for both branches as the maximum value of the force in all cycles for each displacement level [1].

Figure 6 depicts the variation of secant stiffness in each phase of the test with respect to the initial stiffness as a function of the maximum displacement reached in each phase. In the positive branch it is observed that the degradation is less abrupt compared to the negative branch and is related to the ductile behavior observed in both specimens. In this case, the stiffness degradation is the result of the gradual cracking and the loss of concrete rebar bond [2].
In the negative branch it is observed that the stiffness degradation is abrupt and is associated with brittle failure due to cracking of the concrete without rebars in the beam-wall interface zone. This is mainly observed in the specimen CB-1 for the complete absence of any kind of reinforcement.

Before the brittle failure occurs in the negative branch, in the hysteresis curves of both specimens a great reduction in stiffness is observed during the recharge towards the negative branch, after the discharge of the positive branch. This is most clearly seen in specimen CB-2. This phenomenon called pinching is the result of the partial recovery of stiffness when cracks close during displacement in the opposite direction to that which produced these cracks [2]. After the failure occurs in the negative branch, the stiffness is zero. The trajectory in the negative branch is completely horizontal.

In the negative branch it is observed that the strength degradation occurs suddenly. This loss of strength is associated with the brittle failure that occurs in this test direction. The stiffness and strength degradation is observed mainly in specimen CB-1 due to the complete absence of any kind of reinforcement. In the negative branch, the strength degradation is produced by the repetition of the cyclic displacement as a function of the total energy demanded and by the increase in the inelastic displacement as a function of the demanded ductility [2].

3.3 Energy dissipation capacity

The energy dissipation capacity or fracture energy is obtained by calculating the area enclosed by the hysteretic loops in each of the cycles. Figure 7 [a] shows the contribution of the positive and negative branch to the dissipated energy and Figure 7 [b] shows the total accumulated dissipated energy, both as a function of the maximum displacement reached in each displacement level.

The energy dissipated by the specimen CB-1 in the negative branch is negligible compared to the total energy dissipated. The negative branch contributes 4.6% of the to
Figure 7. [a] Dissipated energy [b] Accumulated dissipated energy

total accumulated dissipated energy. The energy dissipated by specimen CB -2 in the negative branch is appreciable compared to the total energy dissipated, but only up to the failure point. The negative branch contributes 10.3% of total accumulated dissipated energy.

3.4 Performance parameters

The Component Methodology developed in the FEMA P-795 report is a procedure for evaluating new component test records based on performance parameters [1]. The parameters determined in the cyclic test are defined below. The definitions of the parameters are represented graphically in Figure 8:

- The strength \( Q_{ul} \) is the maximum force reached during the test.
- The initial stiffness \( K_i \) is calculated as the secant stiffness corresponding to the force and displacement at 40% of the maximum force \( Q_{ul} \).
- The effective yield strain \( \Delta y_{eff} \) is calculated as the maximum force divided by the initial stiffness \( \Delta y_{eff} = \frac{Q_{ul}}{K_i} \).
- The ultimate strain \( \Delta u \) is the strain associated with the drop in strength up to 80% of the maximum force \( Q_{ul} \).
- The effective ductility capacity \( \nu_{eff} \) is calculated as the ultimate strain divided by the effective yielding strain \( \nu_{eff} = \frac{\Delta u}{\Delta y_{eff}} \).

The performance parameters calculated according to the definitions given in the FEMA report P-795 [1] are shown in tabular form in Tables 1 and 2 for the positive branches [+] and negative [−] respectively.

### Table 1. Performance parameters in the positive branch [+]

<table>
<thead>
<tr>
<th>Spec</th>
<th>Strength ( Q_{ul} [kN] )</th>
<th>Stiffness ( K_i [kN/mm] )</th>
<th>Ductility ( \nu_{eff} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-1</td>
<td>271.63</td>
<td>49.89</td>
<td>14.09</td>
</tr>
<tr>
<td>CB-2</td>
<td>265.16</td>
<td>60.25</td>
<td>22.77</td>
</tr>
</tbody>
</table>

### Table 2. Performance parameters in the negative branch [-]

<table>
<thead>
<tr>
<th>Spec</th>
<th>Strength ( Q_{ul} [kN] )</th>
<th>Stiffness ( K_i [kN/mm] )</th>
<th>Ductility ( \nu_{eff} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-1</td>
<td>73.47</td>
<td>73.47</td>
<td>1.26</td>
</tr>
<tr>
<td>CB-2</td>
<td>80.48</td>
<td>73.99</td>
<td>20.74</td>
</tr>
</tbody>
</table>

In the positive branch, the average values of the strength, initial stiffness and ductility parameters are 268.4\( kN \), 55.1\( kN/mm \) and 18.4 respectively. The ultimate displacement reached by specimen CB-2 is 30.7% higher than the value reached by CB-1. In this branch, the failure of both specimens is caused by the crushing of the compressed concrete. In the negative branch, the average values of the strength, initial stiffness and ductility parameters are 77.0\( kN \), 73.7\( kN/mm \) and 11.0 respectively. In this branch, the failure of both specimens is caused by the cracking of the concrete in tension. In the case of the CB-2 specimen, the post-peak stiffness and strength degradation is less abrupt due to the presence of the CFRP sheets.

3.5 Shear force strength

To analyze the cracked coupling beam-wall interface shear strength, a monotonic test consisting of apply force to a section located 300\( mm \) from the coupling beam-wall interface in a monotonically increasing manner controlled by displacement is performed.

Prior to the test, a 20\( mm \) wide crack is generated by applying vertical force with a hydraulic jack at the free end of the beam.

The maximum shear force achieved is 355\( kN \). At this point no signs of new cracks are seen in the specimen. The initial crack width 20\( mm \) at the top of the beam depth increases by 1.39\( mm \) at the end of the test.

4 Numerical modeling

4.1 Geometry

The model geometry is developed in a three-dimensional space following the dimensions of specimen CB-1. Some simplifications are introduced to reduce the computational demand of the model solution, these are detailed below:
The geometry and boundary conditions of the model are symmetric with respect to the plane passing through half the thickness of the coupling beam-wall sub-assemblage. In this way, by restricting the degree of freedom perpendicular to the plane of symmetry only half of the geometry has been developed.

- The changes in the specimen’s cross section have not been considered.
- Deep changes in the clamping and force application zones are modified from 80 mm and 75 mm to 50 mm to produce a regular mesh.
- Lifting holes are not included in the model geometry. The geometry of the model is represented with linear isoparametric three-dimensional elements of characteristic length equal to 50 mm. The concrete in the sub-assemblage is modeled with 8-node CC IsoBrick hexahedron solids, the reinforcing bars with 2-node CC IsoTruss truss elements, and the rigid force application head with 4-node CC IsoTetra tetrahedra solids. Figure 9 depicts the element type distribution used to represent the geometry.

4.2 Constitutive models

4.2.1 Concrete

The behavior of concrete is represented by the plasticity-fracture model CC3DNonLinCementitious2. The plasticity model is based on the Menetrey-William failure surface, along with elliptical pre-peak and linear post-peak laws.

The concrete compression strength \( f_c = 27.5\text{MPa} \) is determined from test specimens obtained during the construction of the specimens. For Poisson ratio, the value \( \nu = 0.2 \) is adopted, as suggested by the Model Code 2010 [3].

The modulus of elasticity of concrete \( E_c \) is obtained from relation 19.2.2.1 (equation 4.2.1) of the ACI 318-19 [4].

\[
E_c = 4700 f_c = 24647\text{MPa}
\]

The plastic deformation at the point of maximum strength of the concrete \( \varepsilon_c^p = 0.001245 \) is calculated as the difference between the total deformation \( \varepsilon_c = 0.00236 \) (this value is obtained from core tests) and the elastic deformation \( \varepsilon_c^e = 0.001115 \).

The stress fracture model is based on the classical orthotropic distributed crack formulation and the crack band model. The Rankine failure criterion is applied, along with linear pre-peak and exponential post-peak laws. The concrete tensile strength \( f'\tau \) is obtained from the relationship 19.2.3.1 (equation 4.2.1) of the ACI 318-19 [4].

\[
f'\tau = 0.62 P f'c = 3.25\text{MPa}
\]

The fracture energy is calculated using the relation 5.1-9 (equation 4.2.1) of the Model Code 2010 [3].

\[
G_f = 73 f'c^{0.18} = 0.133\text{N/m}
\]

To determine the shear strength of cracked concrete, the aggregate maximum nominal size is considered equal to \( d_{ag} = 20\text{mm}(3/4") \).

For the rest of the concrete parameters, the default values suggested in ATENA software are assumed.

4.2.2 Rebars

The behavior of the rebars is described with the discrete reinforcement model CC Cycling Reinforcement and a multi-linear law. The yield strain is determined to be \( \varepsilon_r = 0.0025 \) and the yield stress to be \( \sigma_r = 450\text{MPa} \). Hardening starts at strain \( \varepsilon_s = 0.0117 \). The ultimate stress is determined equal to \( \sigma_u = 800\text{MPa} \). The cyclic behavior is represented using the Menegotto-Pinto model. For the rest of the steel parameters, the default values suggested by ATENA software are assumed. The prestressing cables are modeled as unbonded internal cables with fixed end points.

4.3 Boundary conditions

4.3.1 Dirichlet boundary conditions

Figure 11 shows the constrained faces. Displacements in the \( u_z \) direction are constrained on the symmetry face. In the \( u_i \) direction, only one face in the fixing zones is constrained to allow the prestressing deformation.

4.3.2 Neumann boundary conditions

To allow the application of force in both directions without generating tensile stress or stress concentration in the concrete, a rigid elastic element has been included. The loading protocol is similar to the one used in the experimental analysis, but in the negative branch only 5 mm displacement is reached, as can be seen in Figure 10).
5 Numerical results

5.1 Crack pattern

Figures 12 [a] and [b] represent the crack pattern at the failure point of the positive and negative branch obtained from numerical model.

In the positive branch, the ultimate failure in the numerical model occurs when the concrete compression strength (27.5MPa) is exceeded in the compressed zone of the beam-wall interface. This same type of failure is observed in the experimental test.

In the negative branch the ultimate failure occurs when the tensile strength of the concrete (3.25MPa) is exceeded at the end without reinforcing bars of the beam-wall interface. This same type of failure is observed in the experimental test.

During the experimental tests no signs of necking, buckling or shearing are observed in the reinforcing bars anchored in the wall, this agrees with the value of the ultimate deformation of 5.26% obtained numerically.

5.2 Hysteretic behavior

Figure 15 compares the hysteresis curve and the cyclic envelope of the numerical results with respect to the experimental result (specimen CB-1).

The pinching effect that is observed experimentally is also present in the hysteresis curve of the numerical model. In the negative branch the numerical model shows the sudden degradation of stiffness and strength observed experimentally. This loss of stiffness and strength is associated with brittle failure that occurs after the elements at the beam-wall interface exceed the tensile strength of the concrete. In the numerical model, brittle failure occurs prematurely in the first cycle of the displacement amplitude of 1mm In the test failure occurs in the first cycle of the amplitude of 2.5mm.

In the negative branch, the degradation of the resistance in the numerical model starts in the first cycle of the displacement amplitude of 75mm in a similar way to the experimental result, but with lower values of displacement and force. In the numerical model, greater initial stiffness
The energy dissipated in the model in the negative branch is negligible compared to the total energy dissipated in a similar way to the experimental result. The energy accumulated up to the displacement amplitude of 50mm is 8358J in the numerical result and 11298J in the experimental one.

### 5.4 Performance parameters

The performance parameters according to the FEMA P-795 [1] obtained numerically and experimentally are compared in Tables 3 and 4 for the positive and negative branches respectively.

#### Table 3. Performance parameters in the positive branch

<table>
<thead>
<tr>
<th>Spec</th>
<th>Strength $Q_M$ [kN]</th>
<th>Stiffness $K_I$ [kN/mm]</th>
<th>Ductility $\mu_{e\ell}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>num</td>
<td>246.18</td>
<td>89.81</td>
<td>26.77</td>
</tr>
<tr>
<td>exp</td>
<td>271.63</td>
<td>49.89</td>
<td>14.09</td>
</tr>
</tbody>
</table>

#### Table 4. Performance parameters in the negative branch

<table>
<thead>
<tr>
<th>Spec</th>
<th>Strength $Q_M$ [kN]</th>
<th>Stiffness $K_I$ [kN/mm]</th>
<th>Ductility $\mu_{e\ell}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>num</td>
<td>53.49</td>
<td>53.49</td>
<td>1.21</td>
</tr>
<tr>
<td>exp</td>
<td>73.47</td>
<td>73.47</td>
<td>1.26</td>
</tr>
</tbody>
</table>

In the positive branch, the strength $Q_M$ and ultimate strain $\Delta U$ obtained from the numerical model are 9% and 4% lower than the experimental values. The initial stiffness $K_I$ in the numerical model is 1.8 times the value calculated with the experimental results. An inverse difference is observed in the yield strain $\Delta_{Y\ell}$ due to the inverse relationship with $K_I$.

In the negative branch, the greatest difference between the numerical and experimental values corresponds to the resistance parameter $Q_M$, the numerical value is 27% less than the experimental value.

### 6 Conclusions

- CFRP sheets increase the resistance and ductility of the coupling beam in the negative branch, but not significantly when compared to the capacity demanded in the positive branch.
- CFRP sheets do not have sufficient ductility capacity to directly reinforce elements with high deformation demands such as coupling beams.
- The contribution of the coupling beams with non-anchoring rebars is not negligible and must be considered as part of the structural system of the buildings.
- The calibrated numerical models adequately represent the behavior of coupling beams and can be used to evaluate numerically other coupling beam configurations in order to improve the estimation of the global performance of buildings.
References


[4] A.C.I. ACI, Building Code Requirements for Structural Concrete (American Concrete Institute, 2019)