CYCLIC TESTS OF PRECAST REINFORCED CONCRETE BEAM-COLUMN CONNECTIONS

H. Guerrero¹, J. Escobar², S.M. Alcocer³, and F. Bennetts⁴

ABSTRACT

An extensive experimental program on full-scale, precast reinforced concrete beam-column connections is under development at the National Autonomous University of Mexico, UNAM. This paper discusses the testing results obtained until date. Comparisons among a benchmark monolithic connection and several precast beam-column joints are presented. After testing, the monolithic connection was repaired using carbon fiber reinforced polymer and re-tested. Precast connections were fabricated using different detailing and various levels of confinement and post-tensioning. Preliminary test results reported are compared in terms of strength and deformation capacities. Conclusions, significant to the precast construction industry, have been formulated. The results show acceptable behavior of the precast system.

Keywords: precast system; reinforced concrete precast elements; beam-to-column connections.

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Cyclic Tests of Precast Reinforced Concrete Beam-Column Connections

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ABSTRACT

An extensive experimental program on full-scale, precast reinforced concrete beam-column connections is under development at the National Autonomous University of Mexico, UNAM. This paper discusses the testing results obtained until date. Comparisons among a benchmark monolithic connection and several precast beam-column joints are presented. After testing, the monolithic connection was repaired using carbon fiber reinforced polymer and re-tested. Precast connections were fabricated using different detailing and various levels of confinement and post-tensioning. Preliminary test results reported in this paper are compared in terms of strength and deformation capacities. Conclusions, significant to the precast construction industry, have been formulated. The results show acceptable behavior of the precast system.

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Introduction

Given its advantages over conventional systems (such as higher quality control and quicker construction), reinforced concrete (RC) precast system is an attractive option to building developers [1]. However, because of its limited capacity for moment transfer at beam-column connections, this system has been widely questioned in seismic countries [2]. Although some criticism may be based on bad past experiences, it is apparent that most questions are attributed to lack of knowledge about the system or economic interests from competitors. As a consequence, some seismic design codes (e.g. [3]) impose heavy restrictions on precast construction that effectively lead to discourage its use. If restrictions were to be imposed due to substandard performance, such be based on the capacity of specific connection types and not be generalized to all types. The aim of this investigation is to assess experimentally the seismic performance of distinctly different connection types of precast members. In this paper, main results of an ongoing experimental project at UNAM are presented and discussed. Comparisons among a benchmark

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monolithic connection and several precast beam-column joints are presented. After testing, the monolithic connection was repaired using carbon fiber reinforced polymer and re-tested. Precast connections were fabricated using different detailing and various levels of confinement and post-tensioning. Preliminary test results reported in this paper are compared in terms of strength and deformation capacities. Based on analysis of test data, conclusions, significant to the precast construction industry, were developed.

Experiment setup

Specimens
The tested beam-column connection specimens were sub-assemblages from a prototype four-storey building (Fig. 1a). For convenience, beams were oriented vertically while columns laid down horizontally (Fig. 1b). All specimens had the same geometry, cross-section dimensions, and longitudinal reinforcement (as seen in Figs. 1c-g). The four specimens are described in Table 1, namely: one monolithic connection (specimen 1); two conventional precast connections (specimens 2 and 3); and one post-tensioned precast connection (specimen 4). Note that specimen 1, i.e. the monolithic specimen, was tested twice; as it was retested after rehabilitation using carbon fiber reinforced polymer. It is significant to highlight that stirrup spacing along the beams was varied to analyze its effects on connection performance. In the precast connections (Figs. 1e-g), a precast U-shaped beam was used. Beam bottom longitudinal reinforcement was placed on top of the precast flange. Such position led to a smaller beam effective depth compared to the monolithic connection. The impact of this effective depth reduction will be shown in the results section. The procedure to fabricate the precast specimens is described with the help of Fig. 2. First, the precast parts were fabricated, namely: U-shaped beam and column. Then, the parts were arranged as seen in Fig. 2c. The longitudinal reinforcement was placed (4 bars #8 in the beam’s bottom and 2 bars #12 in the beam’s top), along with the transversal reinforcement ties (#3@300 or 150mm). Finally, the cast-in-place concrete was located.

For specimen 4, post-tensioning was applied using six strands with a diameter of 13 mm (Fig. 2g). A total force of 440 kN (45 ton) was applied, which represents a stress of 1.2 MPa or 2.2% of the concrete compressive strength.

As already mentioned, specimen 1 was repaired after testing. One layer of fiber in the longitudinal direction and one in the transversal direction. While the former one had a length of two effective depths ($d_e$), the latter one was put in a length of 1.5 effective depths. These lengths were considered to cover most of the plastic hinge cracks - formed during the first test and occurred within one effective depth.

Specimens were made of ready-mixed concrete with an average compressive strength of 55.2 MPa obtained from cylinder tests sampled during concrete casting. They were reinforced with low-carbon deformed bars with a specified yield stress of 420 MPa.
Figure 1. Specimen characteristics.

Table 1. Tested specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test</th>
<th>Description</th>
<th>Stirrups spacing, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>Monolithic (benchmark)</td>
<td>300</td>
</tr>
<tr>
<td>1</td>
<td>1-R</td>
<td>Monolithic repaired</td>
<td>300</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>Precast 1</td>
<td>300</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>Precast 2</td>
<td>150</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>Precast Post-tensioned</td>
<td>150</td>
</tr>
</tbody>
</table>

Figure 2. Fabrication procedure for precast specimens.
**Instrumentation**

Specimens were instrumented to assess reinforcement strains, member deformations and displacements (see Fig. 3). Linear variable differential transformers (LVDTs) were installed to measure local deformations and displacements as follows: 1) horizontal LVDTs were placed at different positions along the beam; 2) vertical LVDTs on the beam and the column to measure rotations; and 3) diagonal LVDTs to measure shear deformations. Electric foil strain gauges were adhered to the concrete surface and bar reinforcement to assess strain distributions (see Figs. 3d and 3e). A load cell was also located in the actuator to record the applied loads.

![Diagram of Instrumentation](image)

Figure 3. Instrumentation of specimens.

**Loading protocol**

The ACI 374.2R-13 testing protocol was followed [4]. First, small load controlled cycles were applied to the specimens in order to measure initial stiffness. Cycles to 4.3, 6.4, 8.5 and 11 t were applied whilst the expected yielding load was previously calculated at 16 t. Two cycles were applied at each load level. After that, a displacement controlled history was applied. The
displacement controlled cycles applied to each specimen is shown in Fig. 4. The inter-storey drifts, calculated as the ratio between beam tip displacement and beam length, are also shown for illustration purposes. Note that one smaller deformation reversal is incorporated within each pair of cycles at the same displacement level in order to: 1) re-center the specimens before applying larger deformations, and 2) measure the stiffness degradation after each significant deformation demand level.

Results

Load deformation curves

The load-displacement curves are shown in Fig. 4. Displacement is that recorded at the beam tip. The design loads are indicated for illustration purposes. They were calculated using the equations provided by the Mexico City design code [3]. Note that the negative design force is smaller in the precast specimens due to the effective depth reduction of the beams bottom steel reinforcement, as mentioned in the experiment setup section. Some significant observations are worth to be highlighted from Fig. 5, namely: 1) stiffness of the precast specimens was slightly smaller than that of the monolithic connection; 2) tested specimens yielded at displacements between 40 and 50 mm (or equivalent drifts to 0.8% and 1.0%); 3) since the maximum applied displacements were 150 mm (drift of 3%), displacement ductility ratios between 3 and 3.75 were recorded; 4) in all cases, measured strengths were higher than their corresponding design strengths; and 5) precast specimens exhibited more pinching behavior close to zero displacement levels.

![Load-displacement curves](image-url)
The four specimens’ load-capacity curves are shown together in Fig. 6. In the graph, the applied force has been normalized by their corresponding design shear strengths. It is readily apparent that the lateral load carrying and deformation capacities are similar in the monolithic and the precast specimens.

Figure 6. Normalized load-displacement curves together.

Figure 7. Specimen 1 before and after repair with a CFRP jacket.
As it was indicated above, the monolithic specimen was repaired with carbon fiber reinforced polymer – CFRP - (Fig. 7a to 7c). It is significant to mention that voids and cavities were filled up with high strength mortar \( (f'_c = 70 \text{ MPa}) \) while small cracks (smaller than 2 mm) were injected with epoxy. A comparison of the hysteretic behavior of the monolithic specimen, before and after repair, is shown in Fig. 7d. It can be seen that, even when the level of damage after testing was severe (Fig. 7b), the CFRP jacket was effective to recover, and even, exceed the specimen’s original load-carrying capacity (Fig. 7d). Therefore, it can be concluded that CFRP jacketing is an effective rehabilitation technique for heavily damaged beam elements.

Figure 8. Comparison of physical damage of Specimens 1 to 4 for a displacement of 150 mm.
Comparison of cracking
The level of cracking of specimens for the maximum lateral displacement of 150 mm (equivalent to a drift of 3%) is presented in Fig. 8. Specimen 1 exhibited the largest damage with severe beam flexural and inclined cracking, concrete crushing and incipient buckling of beam longitudinal bars. The precast specimens showed smaller levels of damage, limited to flexural and diagonal cracking. The reduced level of damage is explained by the concentration of the plastic deformation of the beam longitudinal reinforcement at the beam-column interface; where cast-in-situ and precast concrete formed a cold joint. Also note the reduced level of cracking on the post-tensioned element; which is an advantage of this element over the others.

Conclusions
Preliminary results of an ongoing experimental project on precast reinforced concrete beam-column connections were presented in this paper. Four specimens were tested, namely: a benchmark monolithic connection, two traditional precast connections and a precast post-tensioned connection. After been tested, the monolithic connection was repaired and re-tested. The load-deformation curves were presented as well as a comparison of the level of cracking under the maximum deformation of the specimens (equivalent to an inter-story drift of 3%). Based on the preliminary results and data analysis from tests, the following conclusions are offered:

- The precast specimens presented slightly smaller stiffness than the monolithic one.
- In terms of load – deformation hysteresis curves, the monolithic and precast beams had similar behavior. All specimens yielded at an equivalent inter-story drifts between 0.8% and 1%. Measured displacement ductility levels ranged between 3 and 3.75.
- The monolithic and precast connections reached higher load capacities than those calculated using design code equations.
- The precast specimens showed some pinching behavior close to zero displacement levels.
- The repaired monolithic connection using a carbon fiber reinforced polymer jacket was effective to recover its load and displacement capacities even when the original specimen had been severely damaged. CFRP jacket was made of one ply of carbon fiber sheets.
- In terms of cracking, all precast connections exhibited less amount and severity of damages as compared to that of the monolithic connection. The reduced level of damage, i.e. smaller width cracks, is credited to concentration of the plastic deformation of beam longitudinal reinforcement at the beam-column interface; where cast-in-situ and precast concrete formed a cold joint.

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