BEHAVIOUR OF A PRECAST CONCRETE BEAM-COLUMN CONNECTION

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SUMMARY

The behavior of a precast concrete beam-to-column connection was assessed experimentally. Two beam-column joints were constructed and tested under unidirectional and bi-directional cyclic loading that simulated earthquake-type motions. Variables were the type of detailing used at the joint to achieve structural continuity of the beam reinforcement, and the type of construction (two-dimensional or three-dimensional). The most relevant feature of the detail adopted to achieve beam continuity is that neither welding nor special bolts were used. Rather, conventional reinforcing steel bars or prestressing strands were necessary. In one structure, continuity was achieved by placing hoops around the extensions of 90-deg hooks of beam bottom reinforcement that protruded from the beam ends into the joint. In the other specimen, hooks were replaced by U-shaped prestressing strands that were lap-spliced to the bottom beam reinforcement that was in turn terminated flush at the beam end; a steel bar was inserted vertically through the intersection of looped strands. Specimen design criteria follow the strong-column – weak-beam concept. Beam reinforcement was purposely designed and detailed in order to impose large inelastic shear demands into the joint. During specimen fabrication, it was clear the ease, speed and reliability of construction achieved with the joint details tested. Specimen failure was controlled by joint shear. Premature, yet unexpected, inelastic deformation of hoops in the first structure, as well as beam rotation inside the joint in both structures, led to early joint distress. Beam yielding occurred prior to joint failure. Joint strength was 90 percent that expected for monolithic reinforced concrete construction. Specimen behavior was ductile, while strength was maintained at a nearly constant value up to drift angles to 3.5 percent. Stiffness deterioration followed a parabolic decay.

INTRODUCTION

Structural systems based on precast concrete elements have been shown to be safe, durable, reliable and cost-effective. However, their full implementation in seismic design has been limited due to scarce design guidelines compared to reinforced concrete systems. In particular, the lack of development of design provisions of seismic-resistant beam-to-column connections appropriate for these structures is apparent (Vasconez *et al*. 1994). This research focuses on the behavior under inelastic cyclic loading of the connections of precast frames developed by *Servicios y Elementos Presforzados S.A.* (SEPSA) and the National Center for Disaster Prevention (CENAPRED).

The most common criterion for the design of precast concrete lateral force resisting systems is the emulation of monolithic reinforced concrete construction (Ghosh *et al.* 1997). The other alternative is the use of the unique properties of the precast concrete elements interconnected either by dry or wet connections. In the frame under consideration, design was aimed at emulating monolithic construction. It is assumed that beam hinging would develop as the primary source of energy dissipation. This investigation was directed to assess the joint behavior and its stiffness, deformation and strength characteristics when subjected to large shear and bond demands. It was considered that such information would be useful to assess the validity of the emulation hypothesis.

DESCRIPTION OF THE EXPERIMENTAL PROGRAM

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Specimen Details and Design Criteria

Two full-scale beam-column connections made with precast beams and columns were constructed and tested. Specimens represented an interior joint of a lower story of a multistory building. Experimental variables studied were the type of construction (two- and three-dimensional), and the joint detailing for continuity of beam reinforcement. First specimen, J1, consisted of two beams framing into the joint on opposite sides (2D construction); bottom longitudinal steel reinforcement of the beams was terminated with 90-deg hooks at the joint. Continuity of this reinforcement was achieved with hoops placed around the extensions of the 90-deg hooks that protruded from the beam ends. Hoops had a 90-deg bend with a six-diameter extension. Specimen J2 had beams framing into the column from two orthogonal directions (3D construction); bottom longitudinal reinforcement of the beam was interrupted at the joint face and lap-spliced with a U-shaped prestressing strand that extended from the beam into the joint. Continuity was achieved with a steel bolt inserted through the overlapping U-shaped prestressing strands at the joint mid-depth.

Concrete in columns was interrupted at the floor level during fabrication in the precast plant to allow placement of beams. Nonetheless, column longitudinal reinforcement was continuous and was provided in bundles at the corners to leave ample free space for beam erection and placement.

Specimen geometry is shown in Fig.1; specimen J1 is presented in the drawing. Specimens consisted of beams 5660-mm long framing into a column 4140 mm high at mid-height. Column was square with 500-mm sides. J1 had beams with a final 500-mm square section. J2 had beams with a final 500-mm square section in the EW direction and Tee beams with a final 200-by-500-mm section in the NS direction. Dimensions shown correspond to the distance between hinged supports.

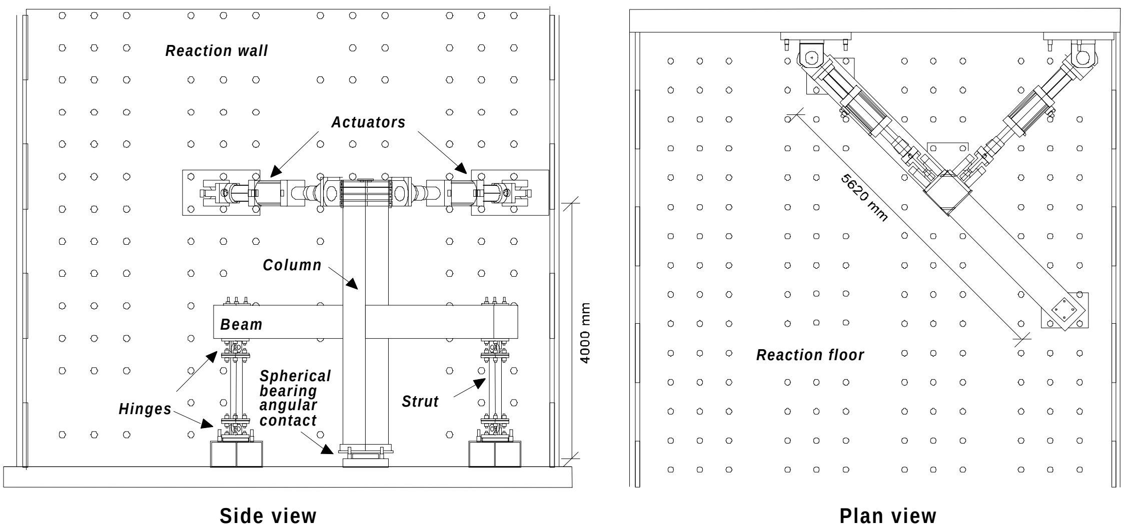

Figure 1. Specimen Geometry (J1)

Where applicable, specimens were designed according to the Mexico City Building Code, MCBC (*Departamento,* 1987), which has similar requirements for special moment-resisting frames to those for monolithic construction in Chapter 21 of ACI 318-95. Although structures were expected to behave in a ductile manner by developing flexural plastic hinges in the beams next to the column, high shear demands in the joint were accepted. This was deemed appropriate for assessing the behavior of the joints under large inelastic deformations. Thus, the longitudinal steel arrangement of columns was designed to reduce the likelihood of yielding, whereas beam reinforcement and detailing were designed to ensure plastic hinging and to impose large shear demands into the joint.

Columns and beams were built in SEPSA′s precast plant. Column and beam reinforcement, and joint detailing of J1 are shown in Fig. 2; joint reinforcement of J2 is presented in Fig. 3.The column of J1 was reinforced with 8 No. 10 continuous longitudinal bars and with No.4 hoops at 100-mm spacing. The height of the gap in the column for beam placement and casting of the joint was 1000 mm. Precast beams had an inverted T-shape and were reinforced with 2 No.8 bottom longitudinal bars and with No.3 stirrups at 100-mm spacing. Bottom bars protruded from the beam ends into the joint with 90-deg hooks. Additional No.5 and No.3 longitudinal bars were placed in the columns and beams to ease the fabrication of the cages.

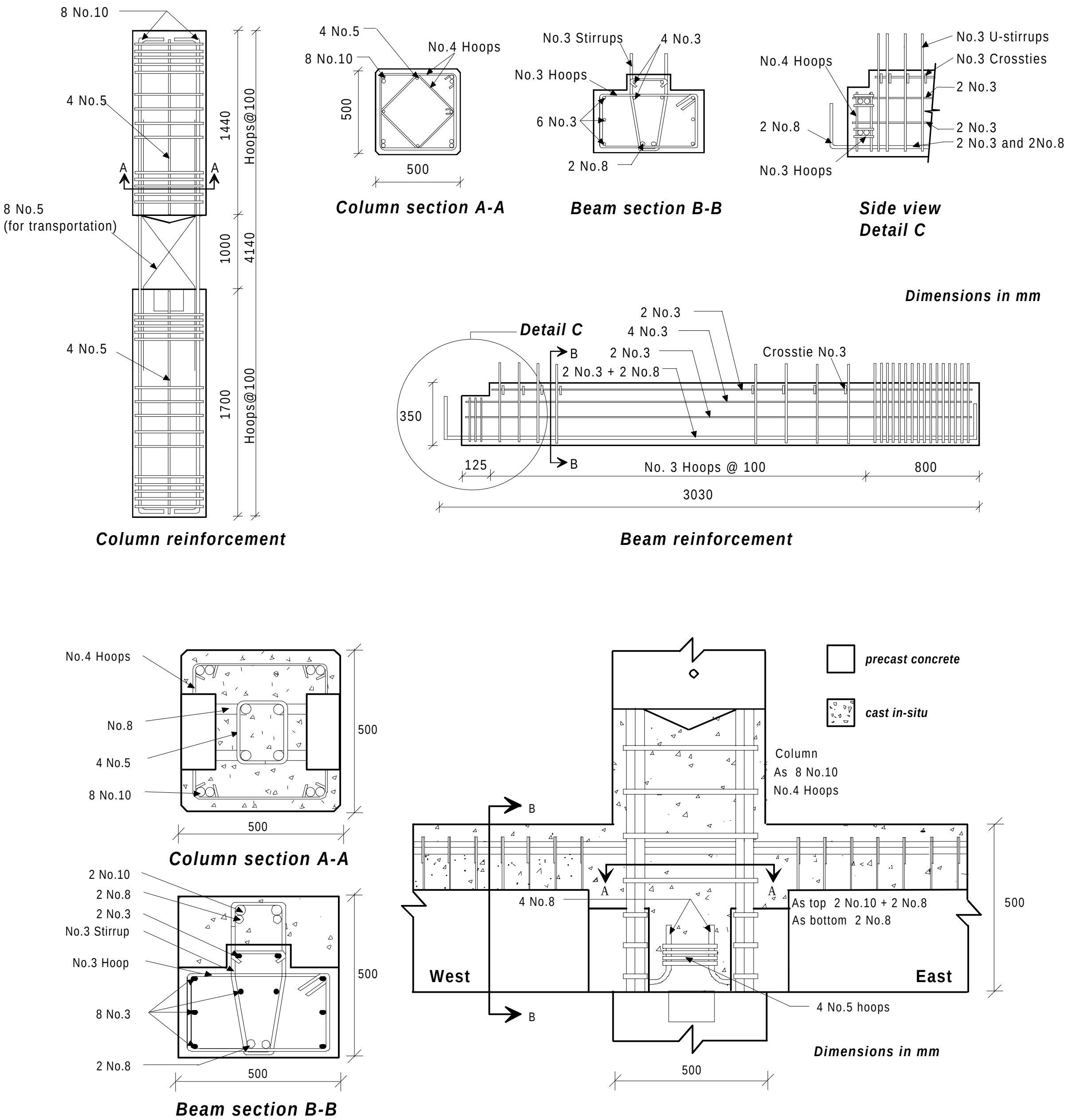

Figure 2. Reinforcement of Specimen J1

The column of J2 had same dimensions and transverse reinforcement as J1; however, continuous longitudinal reinforcement was made of 8 No.12 bars. In the EW direction, beams were similar to those of J1. In the NS direction, Tee-beams were used and were reinforced with 2 No.8 bottom longitudinal bars and with No.3 stirrups at 100-mm spacing. In both, the EW and NS beams, bottom longitudinal bars were interrupted at the joint. To provide continuity, looped (U-shaped) prestressing strands were lap-spliced with the deformed longitudinal bars.

In both specimens, to improve direct shear transfer at the joint between precast and cast *in situ* concrete, a niche was left in the lower side of the column gap. It was intended that during placement of the joint concrete, concrete would fill the niche so that it would work as a shear key. Since beam width in the EW direction was the same as the column width, a 250-mm reduction was made at the joint in order to fit the beam between the column longitudinal bundled bars.

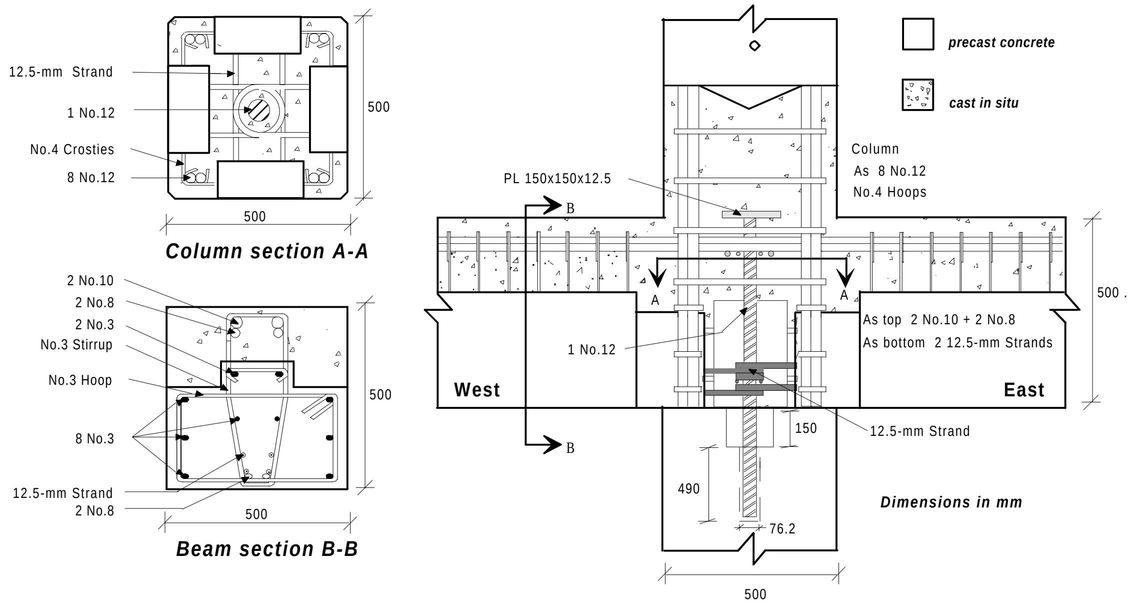

Figure 3. Joint Reinforcement of Specimen J2, EW Direction

Once the column and the beams were mounted on the test rig (Fig.1), continuity reinforcement of bottom longitudinal bars of the beams was placed. For J1, this reinforcement consisted of 4 No.5 hoops placed around the extensions of the 90-deg hooks. Hoops were proportioned to yield while transferring the axial force developed at ultimate along the beam bottom reinforcement. That is, the total area of hoop legs in the direction of loading was similar to the area of beam bottom longitudinal bars.

For J2, a No.12 bar was inserted vertically through the intersection of the looped (U-shaped) prestressing strands that protruded from the beams; the bar was anchored inside the column niche. To improve bar anchorage inside the joint, a square steel plate was butt-welded in the upper part. Prestressing strands were designed to yield when beam bottom bars plastified. The vertical steel bar was designed to remain elastic under yield demands from the strands at ultimate.

To improve the confinement, strength and deformability of the joint concrete, No. 4 crossties were placed around the joint through holes left across the beam width during precast. Crossties were anchored around column longitudinal bars with 90-deg and 135-deg hooks, alternated over the height. Afterwards, continuous beam top reinforcement was placed through the joint; U-shaped beam stirrups, left anchored during precast, were bent around them. For J1 and the EW direction of J2, beam top reinforcement consisted of 2 No.8 and 2 No.10 continuous bars. In the NS direction of J2 bars were 2 No.8 and 2 No. 6. Finally, ready-mix concrete was placed in the joint and at the top of the precast beams. The final depth of the beams was 500 mm.

Deformed bars were conventional grade 60 steel (f_y=420 MPa). Prestressing strands were low-lax grade 270 steel. Average concrete compressive strength for the columns and beams was 31 and 41 MPa for J1 and J2, respectively; for the joints, average concrete compressive strength was 43 and 40 MPa, respectively.

Test Setup and Loading Program

Column ends and beam ends were pinned (Fig. 1). The upper column was connected to the reaction wall through double-action hydraulic actuators. Beams were connected to the reaction floor through steel struts with hinges at their ends. Specimens were tested under a displacement-controlled cyclic load history that was based on the interstory drift angle and that represented a severe load condition for a beam-column joint (Fig. 4). The loading sequence originated from recommendations made by ACI (1997). Three cycles at same drift level were applied. Bi-directional cycles to 1.5, 2.5 and 3.5 percent drift angles were applied in specimen J2. In this specimen, the main loading direction was EW (beam with square cross section). No axial load was applied to the columns since experimental evidence suggests that axial load has no effect in the joint shear strength (Kurose *et al.*, 1998).

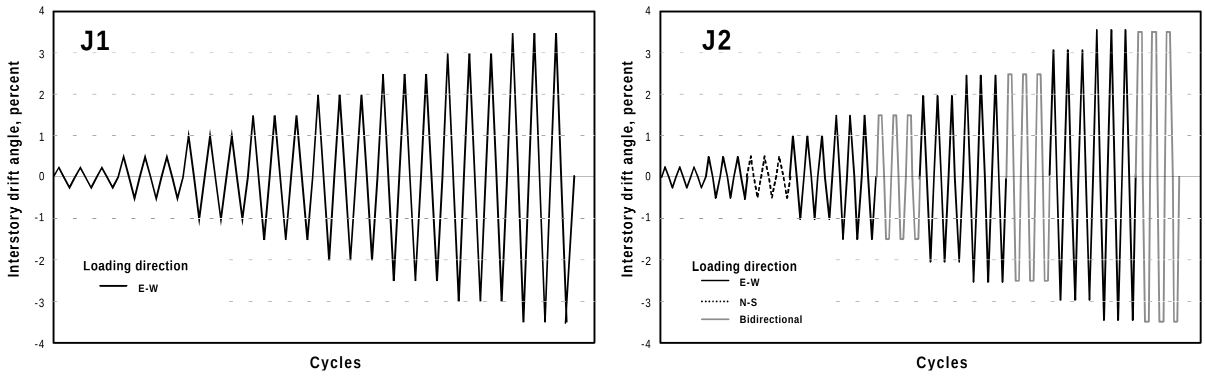

Figure 4. Displacement-Controlled Test Sequence

EXPERIMENTAL RESULTS

Cracking Patterns and Hysteresis Loops

The final crack patterns and the EW-story-shear-versus-drift-angle curves for J1 and J2 are presented in Figs. 5 and 6. Unidirectional and bi-directional cycles are included in the graph of J2. The ultimate story shears corresponding to joint failure $V_{u,j}$ are also shown in the figures. The story shear corresponding to joint failure was based on the shear strength recommended by the ACI-ASCE Committee 352 Report (1991), which is the basis of the requirements in the MCBC (*Departamento*, 1987). Since the joint was not effectively confined on all four vertical sides, a γ- factor equal to 1.25 in MPa units (or 15, in psi units) was used to determine the stress at shear strength, $\gamma\sqrt{f'c}$. Such recommendations are in effect applicable to monolithic construction, but were applied to these cases for comparison purposes. Also indicated in the figures are the occurrence of first joint diagonal cracking and yielding recorded in several locations. The ratio of the calculated column-to-beam flexural strengths, based on measured material properties and dimensions, considering an equivalent rectangular stress block for the concrete and assuming that plane sections remain plane, was 1.63 and 1.94 for J1 and J2, respectively.

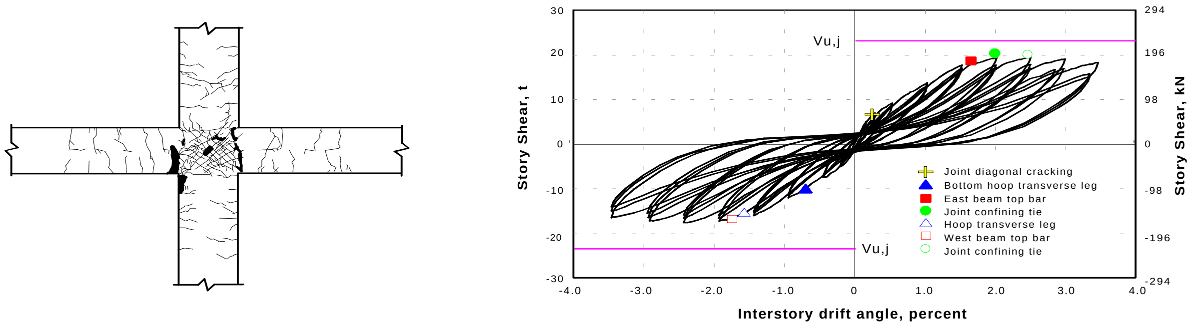

Figure 5. Final Cracking Pattern and Hysteresis Loops for Specimen J1

In both structures, most of the damage was concentrated in the joint and in the beams. Consistent with a "strong column – weak beam" system, column damage was minor. In J1, beams exhibited few flexural cracks near the column face. A 10-mm wide crack was recorded under negative bending moment (top fibers in tension). Under positive bending moment, a 14-mm wide crack was observed at the column face that followed the contour of the precast beam. After demolition, inclined cracking of the narrower portion of the beams was noted. Such damage extended from the interior of the joint, at the lower side of the column gap, up and outs to the beams. This damage is credited to beam rotation inside the joint that was not concentrated at the column face, as it is commonly expected in monolithic construction.

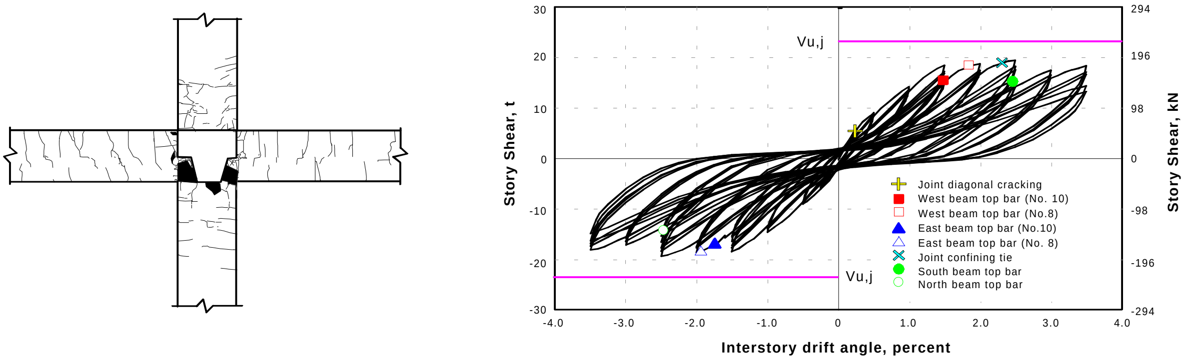

Figure 6. Final Cracking Pattern and EW Hysteresis Loops for Specimens J2

The hysteresis loops are nearly symmetrical and show considerable pinching, especially at drifts to 3.5 percent, and severe stiffness degradation. The hysteresis curves are dominated by the response of the most damaged elements, namely the beams and joint. Analysis of strain-gage data (Pérez-Navarrete *et al*., 1998) indicated that beam top steel bars yielded prior to joint shear failure. Bottom longitudinal steel bars of the beams remained elastic, while the transverse legs of hoops provided for continuity of this reinforcement inside the joint yielded (Fig. 5). Bending flexibility and premature plastification of hoop reinforcement inside the joint led to joint softening and loss of fixity of the beam at the column, thus promoting the concentration of beam rotation inside the joint. From early stages during the test, bending flexibility of hoop legs perpendicular to the loading direction increased beam rotation. Subsequent plastification under bending of such legs further contributed to joint softening (Fig. 7). Undoubtedly, joint behavior was negatively affected by the beam rotation inside the joint. The internal mechanisms of resistance, namely the main diagonal concrete strut and truss actions, and joint stiffness and toughness were impaired by the development of a tensile strain field that cracked the joint concrete early in the test.

The behavior of J2 was superior to that of J1. Beams showed a more uniform distribution of cracking, especially under positive bending (top fibers in compression). Particularly, NS beams exhibited cracking comparable to that observed in monolithic construction (not shown). At similar drift angles, beam crack widths in J2 were smaller than in J1. Nevertheless, inclined cracking in the joint was observed in early stages of the test. As the experiment advanced, damage concentrated in the lower half of the joint, showing a hinge-like distress under positive bending. The hysteresis curves are remarkably symmetrical, even during bi-directional loading. Considerable pinching was observed near the end of the test at drifts larger than 3 percent. Similarly to J1, the hysteresis curves are dominated by the response of the most damaged element, namely the joint. Analysis of strain-gage data during the test indicated that beam top steel bars yielded prior to joint shear failure (Fig. 6); maximum strains recorded were larger than 0.014. Beam bottom longitudinal steel bars remained elastic. The No. 12 bar and the looped (U-shaped) strands also remained elastic. Similarly to J1, beam rotation inside the joint negatively affected its behavior.

The calculated joint strengths $V_{u,j}$ in the EW direction for J1 and J2 were never reached even at cycles to 3.5 percent story drift angles. Measured strengths of J1 and J2 were about 80 and 90 percent, respectively, of those expected in a monolithic beam-column connection designed according to ACI-ASCE Committee 352 (1991).

Response Envelopes

Measured and calculated response envelopes of specimens J1 and J2 are compared graphically in Fig. 8. Measured curves correspond to positive cycles in the EW direction. Beam flexural behavior was assumed to control the specimen response when the envelopes were calculated. The effects of strain hardening of the beam longitudinal reinforcement and of confinement of beam concrete on specimen strength, stiffness and inelastic deformability were taken into account in J1. For J2, two predictions were made; in both calculations, the contribution of strain hardening and confinement was included. The uppermost curve was obtained by assuming that the beam bottom Grade 60 bars and the prestressing strands could be mobilized to reach yielding. The lowest estimation only considers the contribution of strand yielding at the column face. Measured envelopes of J1 and J2 are almost identical. Initial lateral stiffness of specimens was similar, but was half the calculated value. For J1, agreement between measured and calculated behavior is very good up to drift angles to 2 percent. For larger deformations, calculated response showed higher strength and stiffness due to the consideration of strain hardening and concrete confinement. Analysis of experimental data did not reveal evidence of strain hardening of the beam longitudinal reinforcement, nor significant improvement of concrete characteristics in the beams next to the column due to confinement. Measured envelope of J2 is between the predicted curves. It indicates that some of the beam bottom reinforcements (deformed bars and strands) were mobilized. The shape of measured responses can be idealized by a trilinear model defined by flexural cracking at 0.25 percent drift angle, strength at about 1.5 percent drift angle and ultimate “strength” at 3.5 percent. The slope of the branch between strength and ultimate can be assumed to be zero.

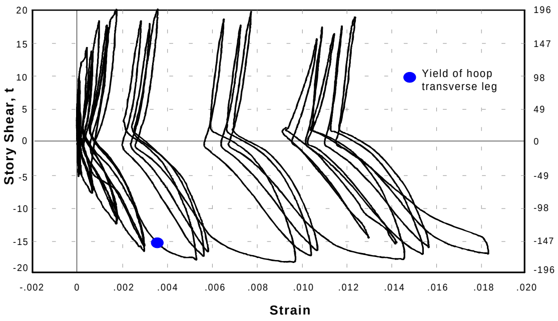

Figure 7. Strain in Transverse Leg of a Continuity Hoop

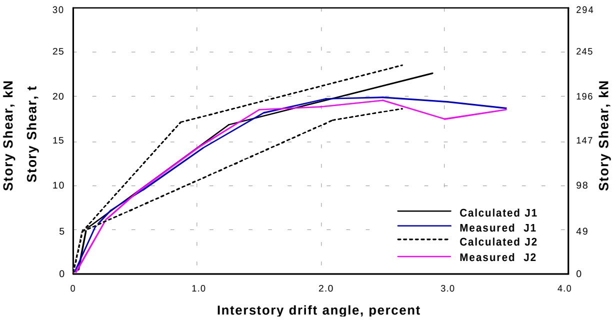

Figure 8. Measured and Calculated Response Envelopes of J1 and J2

Stiffness Deterioration

In order to assess stiffness deterioration, the secant stiffness was computed for each loading cycle. The secant stiffness was calculated using a straight line between the maximum load and corresponding drift-angle points for the positive and negative directions in a loading cycle. Both specimens exhibited a similar rate of stiffness deterioration (Fig. 9). At an interstory drift angle of 2 percent, J1 and J2 had retained 40 percent of their initial stiffness. The different joint detailing used to achieve continuity of beam reinforcement in J1 and J2 did not impact the trend observed.

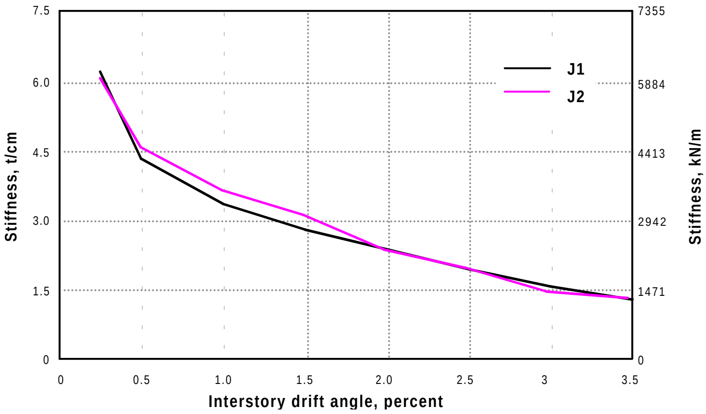

Figure 9. Secant Stiffness of J1 and J2

CONCLUSIONS

Based on the observation and results during construction, testing and data analysis, the following conclusions were developed.

1. The two types of connections tested proved to be efficient and reliable systems that simplify and speed up the construction of precast concrete frame structures.

2. Specimens exhibited a ductile response; the lateral load-carrying capacity was maintained nearly constant up to drifts to 3.5 percent, which are larger than maximum values allowed in most design codes in the world.

3. Specimen behavior was controlled by joint shear. Severe joint strength degradation and stiffness decay were recorded after beam top steel reinforcement had yielded in tension.

4. In both structures, beam rotation took place inside and outside the joint. Joint mechanisms of resistance were impaired by the development of a tensile strain field due to beam rotation inside the joint. Beam rotation inside the joint is not observed in monolithic construction.
5. In J1, where hoops were used to achieve continuity, the bending flexibility of hoop legs transverse to the loading direction contributed to initial joint damage. The plastification of hoop legs affected the internal joint shear-resisting mechanisms.
6. Specimen J2 (where continuity of bottom longitudinal reinforcement of the beams is provided by a steel bar inserted through overlapping U-shaped prestressing strands) performed better than J1. J2 exhibited a more uniform distribution of beam cracking and yielding under negative bending.
7. Continuity reinforcement, either hoops or U-shaped strands should be strong and stiff enough to avoid plastification under maximum demands calculated from a capacity design approach.
8. Joint shear strengths of J1 and J2 were 80 and 90 percent of those expected for monolithic construction. Moreover, initial shear cracking occurred at lower levels of nominal shear stress than in monolithic construction. This phenomenon was credited to premature beam rotation inside the joint.
9. In accordance with the results obtained, connections tested do not properly emulate monolithic construction. However, they can be used in precast concrete frame systems or in dual systems, provided their properties (strength, stiffness) are accounted for.
10. Should emulation of monolithic construction be desired, beam rotations inside the joint should be minimized. One approach to accomplish this objective is to force the concentration of beam rotations far from the column faces, i.e. relocate the beam plastic hinges.

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