An experimental study of the behavior of precast concrete beam-to-column connections subjected to cyclic inelastic loading conducted at the National Institute of Standards and Technology is presented. The study was initiated to provide data for the development of a rational design procedure for such connections in seismically active regions. The objective of the study is to develop a moment resistant precast concrete connection that is economical and can be easily constructed. Four one-third scale monolithic concrete beam-to-column connections were tested: two were designed according to the 1985 Uniform Building Code (UBC) Seismic Zone 2 criteria and two to UBC Zone 4 criteria. In addition, two precast, post-tensioned concrete beam-to-column connections similar in design to the monolithic Zone 4 specimens were tested. These tests constitute the first phase of a multi-year test program.
Many experimental and analytical studies have been conducted in the past on the performance of reinforced monolithic concrete beam-to-column connections subjected to cyclic inelastic loadings. However, there have been only a limited number of studies on the performance of precast concrete connections and into a lesser extent on moment resistant precast concrete beam-to-column connections. This is true even though precast concrete construction has been in use in the United States since the 1950s.

Due to the limited data available, it has been presumed that precast concrete structures tend to be less ductile and tend to have a less stable inelastic response than cast-in-place structures. This is primarily due to the concentration of inelastic strains in the connection regions. Consequently, only general provisions for the design of precast concrete structures have been included in American building codes [e.g., the Uniform Building Code (UBC)]. Since the UBC is the most commonly used or referenced code in seismically active areas in the United States, precast construction is less prevalent in these regions.

The need for a more comprehensive guideline for precast concrete structures has been recognized by both designers and researchers. A workshop conducted by the Applied Technology Council on the design of prefabricated concrete buildings for earthquake loads had been held in 1981 to determine current knowledge of precast concrete structures and to identify research needs. Forty research areas were identified and the topic receiving the highest priority was one which called for the development of a recommended practice for moment resistant beam-to-column connections.

In response to these needs, a study of the behavior of precast concrete beam-to-column connections subject to cyclic inelastic loading was initiated at the National Institute of Standards and Technology (NIST) in 1987. The goal of the experimental program was to develop recommended guidelines for the design of precast beam-to-column connections in seismically active regions. Emphasis was placed on an economical and easily constructible connection since economics is a key consideration in the undertaking of any construction project.

The overall test program involved the testing of one-third scale model interior beam-to-column connections. This scale was selected as a result of the size limitations imposed by the test facility at NIST. The experimental program consists of three phases.

The first phase of the experimental program included the tests of two monolithic concrete beam-to-column connections designed in accordance with 1985 UBC Seismic Zone 4 criteria and two monolithic beam-to-column connections designed in accordance with UBC Seismic Zone 2 criteria. The design was based on the 1985 instead of the 1988 UBC requirements because the former edition was in use at the time the test program was initiated. Results from these tests were intended to provide a reference for comparison with later precast concrete connection tests. In addition, two posttensioned precast concrete specimens designed similarly to the monolithic Zone 4 specimen were tested. Zone 4 will be taken to mean UBC Seismic Zone 4 and Zone 2 will be taken to mean UBC Seismic Zone 2 throughout this paper.

The second phase of the project will involve testing of three sets of posttensioned beam-to-column specimens. Each set will consist of two identical specimens designed to investigate the effects of location and distribution of the prestressing tendons and to determine if any difference in performance results from using prestressing strands instead of post-tensioning bars.

Phase III of the program will consider methods of improving the hysteretic behavior of the precast concrete specimens and the presence of precast concrete slabs on the strength and ductility of the connections. Some areas in this phase will be coordinated with the Precast Seismic Structural Systems (PRESSS) project which is part of the U.S.-Japan large scale testing program funded by the National Science Foundation and by the Precast/Prestressed Concrete Institute (PCI). In brief, the intent of PRESSS is to develop comprehensive design recommendations based on research data for precast concrete construction in various seismic zones.

Prior to presenting the main results of Phase I of the test program, a literature review is given. A detailed description of the test program and test results may be found in Reference 4.

LITERATURE REVIEW

In recent years, several studies have been conducted on the behavior of precast beam-to-column connections designed to resist earthquake loads. This section will briefly describe the connection details and results. More detailed information on each of these studies is available in the references listed at the end of the paper.

New Zealand Studies

A study on the behavior of ten monolithic interior concrete beam-to-column connections was conducted by Park and Thompson at the University of Canterbury. Of the ten specimens, one specimen was conventionally reinforced, five specimens had fully prestressed concrete beams, and four specimens had partially prestressed concrete beams. The amounts of prestressed and nonprestressed steel were variables in this study. The specimens were designed so that hinging occurred in the beams at the column face. The columns were subjected to an axial load and the beam ends were subjected to reversed vertical loads.

Results from these tests showed that the flexural strength of the beams exceeded the theoretical strength by an average of 7 percent. A reduction in strength and stiffness was more pronounced in the specimens with the fully prestressed concrete beams than in the specimens with the partially prestressed beams due to the presence of nonprestressed steel in these specimens. Performance of the connection regions was better for the fully prestressed specimens. The presence of a prestressing tendon at mid-depth of the beam was found to improve the performance of the connection region.

Bull and Park tested three full-scale exterior precast beam-to-column connections at the University of Canterbury. The specimens were composite.

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connections consisting of a precast prestressed beam shell with a cast-in-place (CIP) concrete core. Specimens 1 and 3 were detailed for seismic loads while Specimen 2 was not. The difference between Specimens 1 and 3 was the bonding of the interface between the beam shell and the CIP concrete in the plastic hinge region. Specimen 3 was debonded over a length equal to the depth of the CIP beam core.

The columns were subjected to an axial load and the specimens were tested cyclically. Satisfactory behavior was defined as the retention of 80 percent of the specimen strength after four cycles at 4Δy. Yield displacement, Δy, was defined as 1.33 times the displacement of the beam end measured at 75 percent of the theoretical ultimate strength.

Specimens 1 and 3 performed satisfactorily in terms of strength, ductility and energy dissipation and could, therefore, be used in ductile seismic moment resisting frames. The debonded specimen (No. 3) had a longer plastic hinge length in the CIP concrete than Specimen 1. The precast concrete shell in Specimen 3 sustained no damage while the precast concrete shell in Specimen 1 sustained extensive cracking. Specimen 2 experienced sliding shear displacements in the beam at the column face and small energy dissipation.

**Canadian Studies**

Three studies conducted at the Royal Military College were on welded precast concrete beam-to-column connections. These studies were carried out by Pillai and Kirk, Bhatt and Kirk, and Seckin and Fu. The study by Bhatt and Kirk was a continuation of Pillai and Kirk’s work in which the connection details were modified to prevent a weld fracture as experienced in Pillai and Kirk’s study.

In Pillai and Kirk’s and Bhatt and Kirk’s studies, beam reinforcement was welded to column reinforcement and embedded beam plates were welded to embedded column plates. The connections in Seckin and Fu’s work were made by welding two sets of embedded beam plates to two sets of embedded column plates. One set of plates was used to resist flexural stresses and the other to resist shear stress. Flexural reinforcement in the beam was welded to the beam flexural plates.

In these three studies, the connections were subjected to an axial load and to cyclic loading. Comparisons of the welded connections with similar connections cast monolithically indicated that these welded connections performed as well as the monolithic specimens in terms of ductility, energy dissipation, and stiffness.

**American Studies**

In a study sponsored by PCI, eight moment resistant and eight simple connections were tested by Stanton et al., at the University of Washington. The objective of the program was to identify economical and competitive methods in designing precast concrete connections. Only the results of the moment resistant connections are applicable to seismic design and are presented.

The moment connections consisted of a welded connection (BC15), a combination of a cast-in-place topping and a welded connection (BC16A), a bolted column-to-column connection (BC25 & CC1), a precast beam constructed into a CIP column (BC26), a post-tensioned connection (BC27), a connection grouted or partially grouted to dowels (BC28 & BC29), and a composite connection consisting of a precast beam shell filled with CIP concrete using post-tensioning bars as a means of attachment (BC99). The specimens were two-third scale models of prototype connections. Some of the specimens were tested monotonically and some were tested cyclically. A 0.04 radian rotation was defined as the minimum requirement for a ductile frame.

The results from these tests indicated that the dowel connections (BC28 and BC29) could not be classified as moment connections, Specimens BC16A and BC27 could possibly be used in Seismic Zones 1 and 2, and Specimens BC26 and BC99 could be used in Seismic Zones 3 and 4.

A series of seven precast concrete beam-to-column connections were tested at the University of Minnesota by French et al. The connection details varied from post-tensioning with two post-tensioning bars (BMA), a connection using four threaded reinforcing bars (BMB), a composite connection (BMC) consisting of a CIP topping and a precast concrete beam connected with a post-tensioning bar, a welded connection (BMD), a bolted connection (BME), a connection with four threaded reinforcing bars which were threaded into couplers anchored in the column (BMF), and a connection similar to Connection BMF with the difference being the use of tapered-threaded splices (BMG). Specimens BMA to BMF were exterior connections and Specimen BMG was an interior connection.

Specimens BMA to BMF were designed so that the plastic hinge occurred away from the connection region. Specimens BME to BMG were designed so that the plastic hinge occurred in the connection region. The beams were partially prestressed. The specimens were subjected to reversed cyclic inelastic loads. The load was applied at the beam end.

The specimens with the plastic hinge occurring at the conventionally reinforced joint region showed better energy dissipation characteristics than those with the plastic hinge occurring in the prestressed concrete beams. Specimens BMA to BMF achieved interstory drifts of at least 3.3 percent while Specimens BME to BMG achieved interstory drifts greater than 4 percent. In general, the threaded reinforcing bar connection with the tapered splices and the composite connection appear to be the most likely candidates for use in seismically active regions.

A test program at the University of Michigan conducted by Soubra et al. studied the characteristics of fiber reinforced concrete (FRC) composites and examined the use of FRC in the joint between two precast concrete elements. The specimens were made up of two precast concrete beams connected with a CIP joint. The beam was loaded cyclically at the third points. In this study, the parameters included fiber type, volume of fiber and matrix type — mortar or concrete. Six speci-
mens were tested cyclically. The performance of the FRC joints was measured against that of a joint constructed using regular concrete.

Failure of the specimens was initiated by a single flexure crack which led to the eventual fracture of one or more reinforcing bars in the CIP joint. Conclusions drawn from the study were that FRC joints performed better than joints cast with conventional concrete and that FRC joints with steel fibers performed better than FRC joints with plastic fibers.

Other Studies

The Japanese permit the use of precast concrete connections which have been proven to have acceptable levels of strength, rigidity, and ductility. Joint acceptance is based on unit testing and member testing methods. Procedures for these test methods and classifications of the joints based on the results of these tests are outlined by the Building Center of Japan. As a result of this qualification method, much of the research on the precast concrete framing system in Japan is funded by the designer and/or contractor and is therefore proprietary. Some such systems are discussed in Reference 15.

Several European studies involved the testing of connections with embedded structural steel sections in the precast concrete elements. The specimens were in general tested monotonically. The findings indicated that the precast concrete specimens performed adequately when compared with the performance of companion monolithic specimens. Further details of these tests may be found in the references listed.

Several methods of developing satisfactory moment resistant precast concrete connections have been shown in this review. However, some of these methods involve welding, cast-in-place concrete, and bolting. These procedures increase the construction costs since they require better quality control and tighter construction tolerances and therefore, make a precast concrete alternative less attractive.

**SPECIMEN DESIGN AND TEST PROCEDURE**

The design of the test specimens was guided by the project steering committee comprised of individuals...
from the PCI, Portland Cement Association (PCA), a consulting firm, and academia. It was determined that the connections to be tested have details typical of those normally used for reinforced concrete office buildings constructed in Seismic Zones 2 and 4. As stated earlier, the 1985 edition of the UBC was used instead of the 1988 edition. For the NIST test specimens, this meant that the design base shear calculated using the 1985 UBC requirements was 15 percent greater than the value calculated based on the 1988 UBC requirements.

The subassemblage selected for testing was an interior beam-to-column connection. From the literature review and through discussions with the steering committee, it was determined that a post-tensioned precast concrete connection was a potential moment-resistant connection. The connection details would be kept as simple as possible, i.e., no welding, bolting, corbels, or cast-in-place topping. It would, thus, be economical due to ease of construction and rapid erection and be in keeping with the objective of the test program. Steel angles bolted to the precast concrete column would be used in actual field practice as temporary supports for the precast concrete beams until they were post-tensioned and grouted to the column.

**Zone 2 Specimen Design**

The design criterion for the Zone 2 and Zone 4 specimens was the required base shear. The model beam was 6.7 x 10.0 in. (169 x 254 mm) while the model column dimensions were 8.7 x 10.0 in. (220 x 254 mm). The corresponding dimensions for the prototype were a beam 20 x 30 in. (508 x 762 mm) and a column 24 x 30 in. (610 x 762 mm). The beam reinforcement ratio for the bottom steel, \( p \), was 1.3 percent and for the top steel, \( p' \), was 1.8 percent. The column reinforcement ratio was 2.4 percent. The reinforcement details are shown in Fig. 1.

The transverse reinforcement for the column consisted of rectangular ties spaced at 2 in. (51 mm) center-to-center with two transverse and three longitudinal cross ties. The transverse reinforcement was continued through the connection region. Both beam and column longitudinal reinforcement conformed to ASTM A706/A706 M requirements. The design concrete compressive strength was 5000 psi (34 MPa). The same mix design was used for all Zone 2 and Zone 4 specimens.

<table>
<thead>
<tr>
<th>ZONE 2</th>
<th>ZONE 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>a 10&quot;</td>
<td>18&quot;</td>
</tr>
<tr>
<td>b 10</td>
<td>16</td>
</tr>
<tr>
<td>c 40</td>
<td>41-3/4</td>
</tr>
<tr>
<td>d 46</td>
<td>47-3/4</td>
</tr>
</tbody>
</table>

Fig. 3. Interior beam-to-column subassemblage.
Zone 4 Specimen Design

The model beam was 8 x 16 in. (203 x 406 mm) while the column dimensions were 10 x 18 in. (254 x 457 mm). The corresponding prototype beam dimensions were 24 x 48 in. (610 x 1219 mm) and the column dimensions were 30 x 24 in. (762 x 1372 mm).

The bottom beam reinforcement ratio, \( p \), was 0.7 percent and the top reinforcement ratio, \( p' \), was 0.8 percent. These reinforcement ratios are lower than the ratios for the Zone 2 specimens due to the larger size beams used for the Zone 4 specimens. The column reinforcement ratio was 2.0 percent.

The transverse reinforcement consisted of rectangular ties spaced at 1.3 in. (34 mm) center-to-center with six transverse and two longitudinal cross ties and was continued through the connection region. The monolithic Zone 4 reinforcement details for both the beam and column are shown in Fig. 2. The reinforcement conformed to ASTM A706/A706 M requirements.

The criterion used for designing the Zone 4 precast concrete connections was based on the strength of the monolithic Zone 4 specimens. The beam and column dimensions for the precast concrete specimens were the same as for the monolithic concrete Zone 4 specimens. The longitudinal reinforcement for the precast beams consisted of four #3 bars with one bar in each corner of the tie. The column steel arrangement was the same as for the monolithic specimen. No provisions were made to move the plastic hinge away from the column face.

Two 1 in. (25 mm) diameter post-tensioning bars with an ultimate stress of 150 ksi (1034 MPa) were used to connect the precast concrete beams to the precast concrete column. In this first exploratory phase of the test program, post-tensioning bars were used instead of prestressing strands because prestress losses in short strands would be substantial due to seating losses. The initial post-tensioning load was 128.6 kips (572 kN) which resulted in an initial beam stress of 1008 psi (7 MPa). The losses in the post-tensioning bars were minimal because the post-tensioning load was maintained while the nuts were tightened.

The post-tensioning ducts were corrugated and grouted with a grout having a design compressive strength of 6000 psi (41 MPa). The 1 in. (25 mm) wide construction joint was filled with a fiber reinforced grout. The joints were subjected to high compressive loads and it was felt that the fibers would hold the grout together. One and a half percent by volume of straight 0.75 in. (19 mm) long steel fibers were added to the grout mix. The cross section of the fibers was approximately 0.015 x 0.025 in. (0.4 x 0.6 mm). The design compressive grout strength for the joint was 10,000 psi (69 MPa). The faces of the beams and column were roughened to an amplitude of approximately 0.25 in. (6 mm).

Test Procedure

The specimens were pinned at the column base and roller supported at the beam ends and the column top as shown in Fig. 3. These boundary conditions were chosen to model actual conditions where the moments are approximately zero at midspan of the beam and the column.

The vertical and lateral loads were applied as shown in Fig. 3. Each specimen was first loaded axially to a level of 0.1 \( f_c' A_g \). This is approximately half of the vertical load experienced by an interior column in the third story of the prototype structure. The specimens were then loaded laterally in-plane to 75 percent of the calculated ultimate beam moment in the forward (south)
Fig. 6. Hysteresis curves for the monolithic Zone 2 specimens.

Fig. 7. Hysteresis curves for monolithic Zone 4 specimens.

Fig. 8. Hysteresis curves for precast post-tensioned Zone 4 specimens.
direction and then in the reverse (north) direction. The top column displacements were recorded in each direction. The yield displacement, $\Delta y$, was then defined as the average of the two column displacements divided by 0.75. Although this displacement is the displacement of the column top, it corresponds to yielding in the beam. Fig. 4 shows the test setup and the test facility.

The loading history (see Fig. 5) comprised two cycles at $\pm 2\Delta y$, three cycles at $\pm 6\Delta y$, two cycles at $\pm 10\Delta y$, and finally three cycles at $\pm 12\Delta y$. The loading histories for the monolithic Zone 2 specimens varied slightly in that the specimens underwent three cycles at $4\Delta y$ instead of two cycles. This was because the lateral load in the second cycle at $4\Delta y$ fell below 80 percent of the lateral load in the first cycle at $4\Delta y$. Failure of a specimen was defined as the point at which the applied lateral load was less than 80 percent of the lateral load obtained for the first cycle at $2\Delta y$. The test was conducted under displacement control.

**TEST RESULTS**

All the specimens were subjected to the basic loading history described previously. However, the loading history for the precast concrete specimens was modified due to limits of the test facility. The stroke in one direction was limited to $8\Delta y$. Therefore, subsequent cycles after three cycles at $\pm 8\Delta y$ were $10\Delta y$ or $12\Delta y$ in one direction and $8\Delta y$ in the reverse direction.

The hysteresis curves for the monolithic Zone 2, monolithic Zone 4 and precast concrete specimens are shown in Figs. 6 to 8. The nomenclature for the specimens was as follows: First character is the specimen name, second character — M for monolithic and P for precast, third and fourth characters — Z2 for Zone 2 and Z4 for Zone 4.

**Failure Modes**

The failure modes for the three sets of specimens tested were different. The monolithic Zone 2 specimens failed predominantly in shear in the column connection region as shown in the connection region. The column crack widths were less than 0.06 in. (2 mm) at failure. Fig. 10 shows one of the monolithic Zone 4 specimens at failure.

The precast concrete specimens failed as a result of their inability to sustain higher loads due to localized yielding of the grouted post-tensioning bar. This yielding allowed for an opening of approximately 0.5 in. (13 mm) between the beam and column at failure and crushing of concrete in the beams. As with the monolithic Zone 4 specimens, the precast concrete col-

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*Fig. 9. Representative failure mode of a monolithic Zone 2 specimen.*

*Fig. 10. Representative failure mode of a monolithic Zone 4 specimen.*
stiffer than the monolithic Zone 4 specimens. As shown in Table 1, the post-tensioned specimens were approximately twice as stiff as their companion monolithic specimens and five times as stiff as the monolithic Zone 2 specimens.

**Flexural Strength**

The calculated and measured maximum beam moments are given in Table 2. The calculated values were based on an ultimate concrete strain of 0.003 and actual material properties for the steel and concrete. A factor of 1.25 was applied to the yield stress to account for steel strain hardening for the monolithic specimens. However, no factor was applied to the yield stress for the precast concrete specimens to account for strain hardening because the yield stress of the post-tensioning bars was 148.5 ksi (1024 MPa) and the ultimate stress was 159.7 ksi (1101 MPa). Also, the four #3 bars in the beams were neglected when computing the moment for the precast concrete specimens. The measured maximum beam moments were obtained by multiplying the peak beam load as recorded by the load cell by the lever arm.

The average of the four measured maximum beam moments for the monolithic Zone 2 specimens is 54 kip-ft (73 kN-m). This value is 8.5 percent higher than the calculated ultimate moment. However, it should be

![Fig. 11. Representative failure mode of a precast Zone 4 specimen.](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength of concrete* ( f'_c ) (psi)</th>
<th>Measured yield displacement (in.)</th>
<th>Ultimate displacement ductility ( \mu_d ) (kip/in.)</th>
<th>Initial elastic connection stiffness ( \Delta_0 ) (kip/in.)</th>
<th>Story drift at failure (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-M-Z2</td>
<td>6310</td>
<td>0.359</td>
<td>6</td>
<td>44</td>
<td>4.14</td>
</tr>
<tr>
<td>B-M-Z2</td>
<td>5960</td>
<td>0.371</td>
<td>6</td>
<td>38</td>
<td>4.28</td>
</tr>
<tr>
<td>A-M-Z4</td>
<td>4450</td>
<td>0.263</td>
<td>6</td>
<td>121</td>
<td>3.02</td>
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<tr>
<td>B-M-Z4</td>
<td>4670</td>
<td>0.293</td>
<td>6</td>
<td>103</td>
<td>3.38</td>
</tr>
<tr>
<td>A-P-Z4</td>
<td>5890</td>
<td>0.160</td>
<td>10</td>
<td>204</td>
<td>3.07</td>
</tr>
<tr>
<td>B-P-Z4</td>
<td>6450</td>
<td>0.179</td>
<td>10</td>
<td>216</td>
<td>3.44</td>
</tr>
</tbody>
</table>

* Represents strengths obtained from 4 x 8 in. cylinder tests at the time of specimen testing. The same mix design was specified for all sets. However, each set of specimens was cast at different times.

† The initial elastic flexural stiffness was set equal to the slope of the load-displacement plot for the first cycle to 0.75 \( \Delta_0 \) on the initial excursion.

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa; 1 kip/in. = 0.175 kN/mm.

The ultimate displacement ductility for the precast concrete specimens was conservatively taken as 10 due to the unsymmetric loading. This value is higher than the ultimate displacement ductility, \( \mu_d = 6 \), obtained by the monolithic Zone 4 specimens. This increase in ductility may be a function of the steel location. The post-tensioning bars in the precast concrete specimens were located closer to the beam center than was the mild steel reinforcement in the monolithic specimens.

Although the post-tensioned specimens achieved higher displacement ductilities than the monolithic specimens, the story drifts at failure were essentially equal for both sets of Zone 4 specimens. This was because the precast concrete specimens were

* The ultimate displacement ductility for Specimen A-M-Z2 was reported incorrectly in Reference 4. An explanation for this error is given in Reference 19.
noted that it was the deterioration of the column connection region which led to the eventual failure of the connection and not beam degradation. The calculated ultimate moment for the column using actual material properties with an axial load of 51.15 kips (227.5 kN) is 69 kip-ft (94 kN-m). The measured maximum column moment was 57 kip-ft (77 kN-m) at mid-column.

The average of the four measured maximum moments is 109 kip-ft (148 kN-m) for the monolithic Zone 4 specimens and 134 kip-ft (182 kN-m) for the precast concrete specimens. These values are 12.4 and 17.5 percent higher than the calculated moments for the precast concrete specimens and 134 kip-ft (182 kN-m) for the monolithic Zone 4 specimens. The higher value for the precast concrete specimens as compared with the monolithic specimens could in part be a result of strain hardening of the post-tensioning bars which was not taken into account.

### Joint Stress

The experimental joint shear stresses for monolithic Specimens A-M-Z2 and B-M-Z2 were 1.62 and 1.64 ksi (11 and 1.64 MPa), respectively. These stresses are greater than $20 \sqrt{f'}$ (ksi) which is the maximum recommended joint shear stress recommended by ACI-ASCE Committee 352. A lack of sufficient transverse confinement led to joint failure in the monolithic Zone 2 specimens prior to the onset of beam failure.

The calculated joint shear stress using actual material properties for the monolithic Zone 4 specimens is approximately one-third of the maximum recommended shear stress of $20 \sqrt{f'}$ (ksi). This low joint stress was evidenced by the excellent performance of the joint for the duration of the test. The joint shear stress for the precast concrete specimens is approximately two-thirds of the maximum recommended shear stress. The measured value may be less than the calculated value due to slippage of the post-tensioning bar and/or duct in the column region.

### Energy Dissipation

A comparison of the energy dissipated on a per cycle basis up to the third cycle at $6\Delta_y$ is given in Fig. 12 for all the specimens. The energy dissipated per cycle is defined as the area enclosed by the load-displacement plot. The cumulative energy dissipated at various stages in the test is given in Table 3. The cumulative energy dissipated at a particular ductility level was obtained by summing the per cycle energy dissipated up to that point. The cumulative energy dissipated to the third cycle at $6\Delta_y$ is of interest because the load histories for the precast concrete specimens and the monolithic Zone 4 specimens were identical up to this stage. This is also the stage at which the monolithic Zone 4 specimens failed. The precast specimens did not fail at this stage and underwent additional cycles until failure occurred.

As seen in Fig. 12 and Table 3, the energy dissipated by the post-tensioned specimens was closer in range to the energy dissipated by the monolithic Zone 2 beam-to-column connections.

On a per cycle basis, the energy dissipated by the post-tensioned specimens was approximately 30 percent of the energy dissipated by the monolithic Zone 4 specimens. The average of the cumulative energy dissipated by the post-tensioned connections was approximately 20 percent lower than the energy dissipated to failure by the monolithic Zone 4 specimens.

As stated earlier, the ultimate displacement ductility for the post-tensioned specimens was conservatively considered to be 10. If the ultimate

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**Table 2. Comparison of the flexural strengths.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength of concrete* $f'_c$ (ksi)</th>
<th>Calculated ultimate moment, $M_1$ (kip-ft)</th>
<th>Measured maximum moment, $M_2$ (kip-ft)</th>
<th>Avg. of $M_2/M_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-M-Z2</td>
<td>6310</td>
<td>50.0</td>
<td>51 &amp; 59</td>
<td>1.10</td>
</tr>
<tr>
<td>B-M-Z2</td>
<td>5960</td>
<td>50.0</td>
<td>52 &amp; 55</td>
<td>1.07</td>
</tr>
<tr>
<td>A-M-Z4</td>
<td>4450</td>
<td>97.0</td>
<td>109 &amp; 106</td>
<td>1.11</td>
</tr>
<tr>
<td>B-M-Z4</td>
<td>4670</td>
<td>97.0</td>
<td>109 &amp; 113</td>
<td>1.14</td>
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<tr>
<td>A-P-Z4</td>
<td>5890</td>
<td>114.0</td>
<td>130 &amp; 135</td>
<td>1.16</td>
</tr>
<tr>
<td>B-P-Z4</td>
<td>6450</td>
<td>114.0</td>
<td>136 &amp; 137</td>
<td>1.20</td>
</tr>
</tbody>
</table>

* Represents strengths obtained from 4 x 8 in. cylinders at the time of specimen testing.
† Moments measured at the two column faces of the interior connection.

Metric (SI) conversion factors: 1 ksi = 6.895 MPa; 1 kip-ft = 1.356 kN-m.

**Table 3. Comparison of the cumulative energy dissipation.**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Compressive strength of concrete $f'_c$ (ksi)</th>
<th>Ultimate displacement ductility $\mu_d$</th>
<th>Cumulative energy dissipated to Failure (kip-in.)</th>
<th>$6\Delta_y$ Cycle 3 (kip-in.)</th>
<th>$12\Delta_y$ Cycle 1 (kip-in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-M-Z2</td>
<td>6310</td>
<td>6</td>
<td>186</td>
<td>165*†</td>
<td>–</td>
</tr>
<tr>
<td>B-M-Z2</td>
<td>5960</td>
<td>6</td>
<td>204</td>
<td>212†</td>
<td>–</td>
</tr>
<tr>
<td>A-M-Z4</td>
<td>4450</td>
<td>6</td>
<td>597</td>
<td>597</td>
<td>–</td>
</tr>
<tr>
<td>B-M-Z4</td>
<td>4670</td>
<td>6</td>
<td>543</td>
<td>543</td>
<td>–</td>
</tr>
<tr>
<td>A-P-Z4</td>
<td>58890</td>
<td>10</td>
<td>438</td>
<td>165</td>
<td>507</td>
</tr>
<tr>
<td>B-P-Z4</td>
<td>6450</td>
<td>10</td>
<td>477</td>
<td>181</td>
<td>550</td>
</tr>
</tbody>
</table>

* Cumulative energy dissipated through the second cycle at $6\Delta_y$.
† For purposes of comparison, the energy dissipated in the third cycle at $4\Delta_y$ was not included in this summation as the Zone 4 specimens did not undergo this particular cycle.

Metric (SI) conversion factors: 1 kip-in. = 0.113 kN-m.
displacement ductility was considered to be 12 instead of 10, the cumulative energy dissipated up to the first cycle at $12\Delta y$ would be 507 and 550 kip-in. (57 and 62 kN-m) for Specimens A-PZ4 and B-P-Z4, respectively. The average of these two values is approximately 10 percent lower than the average of the cumulative energy dissipated by the monolithic Zone 4 specimens to failure.

**SUMMARY AND CONCLUSIONS**

In Phase I of the precast concrete beam-to-column connection study at NIST, six specimens were tested. Two of the specimens were monolithic concrete connections designed to UBC (1985) Seismic Zone 2 criteria. The other four specimens were designed to UBC (1985) Seismic Zone 4 criteria. Two of the Zone 4 specimens were monolithic concrete specimens while the remaining two were precast concrete with post-tensioned beam-to-column connections. The precast concrete elements were connected by two post-tensioning bars. The construction joint between the beam and column was filled with a fiber reinforced grout and the post-tensioning ducts were grouted after tensioning.

Results from the monolithic concrete tests are used as a benchmark reference for both present and future precast concrete tests. The objective of the test program was to develop an economical moment-resistant precast beam-to-column connection for seismically active regions. The following paragraphs present the conclusions drawn from the results of the Phase I tests.

1. Failure of the monolithic Zone 2 specimens occurred in the joint region due to a combination of high joint stresses and inadequate confinement. The monolithic Zone 4 specimens failed as a result of beam hinging and deterioration. Failure of the precast concrete specimens was characterized by plastic elongation of the post-tensioning bars and crushing and spalling of the concrete cover in the beams. Joint shear stresses for the Zone 4 specimens were below the recommended value of $20 \sqrt{f'c}$ (psi) while the shear stresses in the monolithic Zone 2 specimens were above the recommended value.

2. The ultimate displacement ductilities for the monolithic Zone 2 specimens were 6. These ductilities corresponded to story drifts of 4.1 and 4.3 percent. The ultimate displacement ductility of 10 for the precast concrete specimens was higher than 6 obtained for their companion monolithic specimens. However, since the precast concrete specimens were stiffer than the monolithic specimens, the story drifts at failure for the Zone 4 post-tensioned and monolithic specimens were almost identical. The story drifts at failure for the Zone 4 specimens ranged from 3.0 to 3.4 percent. The post-tensioned specimens were approximately twice as stiff as the monolithic Zone 4 specimens and five times as stiff as the monolithic Zone 2 specimens.

3. The flexural strength of the precast concrete Zone 4 connections was slightly greater than flexural strength of the monolithic Zone 4 specimens. The measured maximum beam moments were on the average 18 and 13 percent greater than the calculated moments (as defined previously) for the precast concrete specimens and the monolithic Zone 4 specimens, respectively. The monolithic Zone 2 specimens achieved measured maximum beam moments that were on the average 8 percent greater than the calculated moment.

4. When comparing the energy dissipated per cycle, the behavior of the precast concrete specimens was closer to that of the monolithic Zone 2 specimens than to the monolithic Zone 4 specimens. On a per cycle basis, the post-tensioned specimens dissipated about 30 percent of the energy dissipated by the monolithic Zone 4 specimens. However, since the precast concrete specimens achieved higher displacement ductilities than the monolithic specimens, the average of the cumulative energy dissipated up to failure by the post-tensioned specimens was approximately 80 percent of that for the monolithic specimens. The average of the cumulative energy dissipated up to failure by the monolithic Zone 2 specimens was about 33 percent of that dissipated by the monolithic Zone 4 specimens.

**RECOMMENDATIONS AND FUTURE RESEARCH NEEDS**

Based on the results of the NIST Phase I test program, it appears that a
post-tensioned precast concrete beam-to-column connection is as strong and as ductile as a monolithic connection and is a viable connection for high seismic regions. However, the energy dissipation characteristics, per cycle and cumulative, of the precast concrete connections could be improved. Further means of improving the energy characteristics need to be explored as energy dissipation is an important factor in high seismic regions.

One possible method of improving the energy dissipation capacity of the precast concrete connections is being investigated in Phase II of the NIST test program currently underway. This procedure involves moving the post-tensioning bars closer to the beam center. Also, in this phase, post-tensioning strands will be used to connect the precast concrete elements. The use of strands is more common than post-tensioning bars in practice. As stated previously, a means of improving the hysteretic behavior of the precast concrete specimens by debonding the pre-stressing strands in the column connection region will be examined in Phase III of the test program. The influence of slabs on the connection behavior will also be studied in this phase.

Acceptance of this type of precast concrete connection will require more research data and a better understanding of the connection behavior. Other considerations that require further study include the use of prestressed concrete beams, ungrouted strands, lightweight aggregate, and the influence of transverse beams.

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REFERENCES