Moment Resistant Connections in Precast Concrete Frames for Seismic Regions

The concept of relocating the plastic hinge region of precast concrete frames for construction in seismic regions is examined. Five beam-to-column subassemblies were tested under simulated earthquake-type loading. The connections included steel plates or angles embedded in the columns and beams which would facilitate field erection. The column axial load was held constant during the test while the free end of the beam was subjected to reversed cyclic displacements. The failure modes, strength and ductility of the test specimens using various connection details are compared. It is concluded that properly designed connections with relocated plastic hinges can perform in a satisfactory manner. The reduced welding requirements for these details result in improved economy of construction.

The design of ductile moment resisting beam-to-column connections for monolithic reinforced concrete structures has been studied by many researchers over the past three decades. As a result of these studies, design recommendations for such connections have been developed by ACI-ASCE Committee 352.1

One disadvantage of monolithic beam-to-column connections is that, for any building more than one story high, elaborate formwork and supports are necessary for construction. Precast concrete members, on the other hand, can be fabricated in a plant under controlled factory conditions. The products are of high quality and can be erected quickly. Consequently, precast concrete frames are an economical alternative to cast-in-place constructions, especially when dead load constitutes the dominant loading of the structure and simple connections are used.

For frames subjected to lateral loading, beam-to-column connections must be capable of resisting large forces and displacements. Due to the difficulties in providing such connection details, the development of economical moment resisting connections for precast concrete frames has been identified as one promising area which
requires further investigation. The large amount of research and number of publications that have appeared in recent years clearly indicate the interest of the industry and the engineering profession in addressing some relevant questions.

In accordance with strong-column weak-beam philosophy, most buildings are designed so that flexural hinges are formed at the ends of the beams, near the column faces. This has been the primary objective of most research conducted on beam-to-column connections for precast concrete frames, including the work of Pillai and Kirk, and Bhatt and Kirk. These studies were conducted to develop a ductile connection detail.

Although the behavior of the tested connections was satisfactory, the construction of these specimens requires significant welding of the beam and column reinforcement. The cost and quality control associated with excessive welding diminishes some of the inherent advantages of precast concrete construction. For the ideal connection, therefore, welding — especially field welding — must be minimized.

The concept of relocating the beam plastic hinges away from the face of the column is relatively new. The advantages of such details are the reduction of strength and ductility demands on the connection. In monolithic structures, this will result in a reduction of confining reinforcement in the connection region.

The first investigations of these connection details were carried out in New Zealand. The results of these studies eventually led to the inclusion of guidelines for the relocated plastic hinge concept in the New Zealand Code of Practice for the Design of Concrete Structures. Recent studies by Abdel-Fattah and Wight, and Al-Haddad and Wight indicated their success in relocating the plastic hinge away from the face of the column and into the beams for monolithic frames.

The studies by French et al. were aimed at finding precast concrete connections which would respond favorably to load reversals. The research provided promising connection details, including a threaded reinforcing bar connection and a composite section comprised of precast and monolithic connections. Recent studies have evaluated connections and new systems which can be used in regions of high seismicity, including the use of bonded tendons to provide the connection between the beams and columns.

**RESEARCH OBJECTIVE**

Because earlier tests had indicated that ductile precast beam-to-column connections were cost prohibitive, an alternative approach would be to design precast concrete connections with relocated plastic hinges. This would result in a reduced demand for strength and ductility in the connection region. Therefore, the two objectives of this study were to develop economical ductile moment resistant connections for precast concrete members and to study the effects of locating the plastic hinge away from the beam-to-column connection.

**DESCRIPTION OF TEST SPECIMENS**

Five specimens, as described in Table 1, were tested for this study. The specimens were designed in accordance with the provisions of the ACI Building Code and the latest design recommendations of ACI-ASCE Committee 352. This report contains recommendations for joint shear stresses, transverse reinforcement, and development length of beam and column longitudinal reinforcement within the joint. These guidelines are for monolithic connections. However, since no specific design recommendations exist for ductile precast connections, the recommendations for monolithic frames were used in this study when applicable.

Each specimen is designated by one or two letters followed by a number as listed in Table 1. The first letter indicates the type of construction, P for precast and M for monolithic. The letter R denotes that the plastic hinge was relocated from the face of the column into the beam. The numerals 1 and 2 refer to the two different details used for the precast connections.

The test column had a cross section of 16 x 16 in. (406 x 406 mm) and a height of 10 ft (3.05 m). Column reinforcement consisted of eight No. 8 or No. 7 Grade 60 longitudinal bars and No. 4 Grade 60 closed ties spaced at 3 in. (76 mm) within the joint and spaced at 4 in. (102 mm) elsewhere. The beams were 14 x 24 in. (356 x 610 mm) in cross section and extended 52 in. (1.32 m) from the column face. No. 4 Grade 60 closed stirrups were provided on 4 in. (102 mm) centers for all beams.

Flexural reinforcement for each beam is listed in Table 1. All beams had equal compression and tension reinforcement. Throughout this paper, reference is made to the top and bottom of the beams. These locations were selected as shown in Fig. 1.

The specimens were designed using the strong-column weak-beam concept. The plastic hinges were expected to develop at the end of the beam near the column for Specimens P1 and P2 and at a distance of one beam depth, Precast Specimen PR2.
The specimens with the relocated plastic hinges were designed taking into account several new concepts including recommendations from Abdel-Fattah and Wight. Two beam cross sections were designed, one at the column face and the other at one effective beam depth away from the column face. It was necessary to design these two sections so that when beam hinging occurred at the intended location, no excessive yielding would occur at the column face.

The beam section at the column face was designed to have a nominal flexural capacity of about 25 percent larger than the maximum anticipated acting moment. Fig. 1 helps explain this concept. The tension and compression steel were used for the beams. The intermediate reinforcement was extended a distance of one-and-a-half times the beam depth from the column face. The intermediate reinforcement area was about 35 percent of the tension or compression reinforcement.

The ratio of the nominal moment capacity of the beam section at the desired plastic hinging location to that at the end of the beam should be selected based on the expected moment diagram for the beam. Here, an attempt was made to keep the ratio of the beam capacity at the hinge to the beam capacity at the column face less than or equal to 0.55. A similar ratio has been used and recommended by other researchers. To achieve this ratio, it was necessary to terminate some of the beam tension and compression steel in the specimens with relocated plastic hinges.

The nominal moment capacity of the beam section at the relocated hinging zone was calculated ignoring the intermediate bars. In other words, it was assumed that the extension of the reinforcement for a distance of half the beam depth is not long enough to develop the full tensile strength of these bars.

### Specimen MR

In an attempt to draw comparisons between precast and monolithic frames, a monolithic specimen was constructed and tested as a control specimen (MR). The beam tension reinforcement consisted of four No. 7 bars placed at 3 in. (76 mm) from the extreme fiber. In addition, four No. 6 intermediate bars were symmetrically placed at 6 in. (152 mm) from the mid-height of the beam. In an effort to relocate the plastic hinge, all intermediate and two of the top and bottom beam longitudinal bars were terminated at a distance of one-and-a-half times the beam depth from the column face.

### Specimens P1 and PR1

The connection detail for the top and bottom of the beam was identical for these two specimens. As shown in Fig. 2, the connection for these two specimens consisted of two fabricated steel T-sections embedded in the column. Each T-section had three holes to allow for the passing of column longitudinal bars. Four No. 7 standard 90 degree hooks were welded to the T-sections to provide adequate anchorage of the plate within the joint. The beam end included two large steel angles to which the longitudinal reinforcement was welded. The welds were ⅜ in. (10 mm) flare-bevel welds, 3 in. (76 mm) long on both sides of the reinforcement. The steel angles and T-sections are shown in Fig. 3.

The intermediate reinforcement for Specimen PR1 was provided in the form of U-shaped No. 6 bars. Each bar was placed in a horizontal plane and its base welded to the steel angles with ⅜ in. (10 mm) flare-bevel welds over the full length of the base. After the beam and columns were cast and cured, the beam angles were welded to the column T-sections with ½ in. (13 mm) fillet welds over the full width of the beam to complete the connection.
Specimens P2 and PR2

The connection detail for Specimens P2 and PR2 was developed in collaboration with the engineers of the Tanner Companies and Stanley Structures, both in Phoenix, Arizona. The detail was designed to provide adequate strength and to facilitate actual construction in the field. Details of the connection are shown in Fig. 4.

For the column, a T-section similar to that of Specimens P1 and PR1 was used on the top. However, for the lower side, a straight plate which extended from the column face was utilized. The plate would serve as a beam seat in the field. In addition, the size of the grouted shear key could vary slightly to accommodate field tolerances such as column misalignment or slight variations in beam length.

Each of the top bars in the beam was welded to a steel angle with ⅜ in. (10 mm) flare-bevel welds, 4 in. (102 mm) long on both sides of the reinforcing bar. The ends of these angles were welded to the vertical face of the steel T-sections in the columns with ¼ in. (6 mm) fillet welds all around and the surrounding area was grouted. On the bottom side of the beam, the bars were welded to a rectangular steel plate. The plate was later welded to the column with a ⅜ in. (8 mm) fillet weld.

Due to a lack of availability of larger sized weldable reinforcing bars, the design of the beams for Specimens P2 and PR2 incorporated No. 6 and No. 5 bars for the main reinforcement (Table 1). The intermediate reinforcement for Specimen PR2 was provided in the form of U-shaped bars placed in a vertical plane so that when one leg of the bars was in tension, the other would be in compression.

TEST SETUP AND INSTRUMENTATION

The specimens were tested in a steel reaction frame as shown in Fig. 5. The column portion of the specimens was placed horizontally in the frame. The specimens were held in position by means of steel clevises which were bolted to anchor bolts cast at the ends of the beam and column. The resulting

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Fig. 2. Reinforcement detail for Specimen PR1. Note: 1 in. = 25.4 mm.

Fig. 3. Detail of steel angle and T-section used in Specimens P1 and PR1. Note: 1 in. = 25.4 mm.
pinned connections were intended to represent the inflection points of the frame. Using a hydraulic jack, an axial load of 20 kips (89 kN) was applied to the column. The axial load was kept constant throughout each test.

Cyclic shear was then applied to the free end of the beam. The loading history selected for the tests consisted of one cycle with a maximum displacement of 0.5 in. (13 mm), two cycles at 1 in. (25 mm), two cycles at 2 in. (51 mm), two cycles at 3 in. (76 mm) and two cycles at 4 in. (102 mm). The initial displacement cycle of 0.5 in. (13 mm) corresponds to small drifts in the elastic range, while the repeated cycles at larger displacements were intended to evaluate the ability of the specimens to maintain their load carrying capacity and stiffness under large inelastic deformations. This loading history was followed for Specimens MR, P1 and PR1. However, because of extensive cracking around the beam plates in Specimens P1 and PR1, the loading cycles were modified to apply displacements in ½ in. (13 mm) increments for Specimens P2 and PR2.

Twelve to 14 electrical resistance strain gauges were placed on beam and column longitudinal and transverse reinforcement. Moreover, in order to monitor the joint deformation and beam rotation during the loading sequence, eight displacement transducers were placed on the specimens as shown in Fig. 6. Throughout the tests, loading was temporarily stopped while the load, displacement transducers and strain gauges were automatically read and recorded using a high speed data acquisition system.

**MATERIAL PROPERTIES**

The concrete for this experimental program, obtained from a ready-mix plant, had a 28-day design compressive strength of 4000 psi (28 MPa). The mix proportions per cubic yard of concrete are as follows: aggre-
Fig. 7. Specimen MR after two 2 in. (51 mm) displacement cycles.

Fig. 9. Specimen P1 after two 2 in. (51 mm) displacement cycles.

Fig. 8. Load vs. displacement for Specimen MR. Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm.

Fig. 10. Load vs. displacement for Specimen P1. Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm.

Gate with a maximum size of ¾ in. (19 mm), 1780 lbs (7.92 kN); sand, 1220 lbs (5.43 kN); Type I cement, 500 lbs (2.22 kN); fly ash, 115 lbs (0.51 kN); and water 310 lbs (1.38 kN). The actual concrete compressive strength for each specimen at the time of the test is given in Table 1.

The reinforcement was Grade 60 weldable steel. The following average yield stresses were obtained from coupon tests of the reinforcement:

- No. 4 bars, 77.5 ksi (534 MPa);
- No. 5 bars, 72.3 ksi (498 MPa);
- No. 6 bars, 69.0 ksi (476 MPa);
- No. 7 bars, 65.7 ksi (453 MPa);
- No. 8 bars, 62.7 ksi (432 MPa).

All plates and angles used were Grade A36 steel. The column plates for the four precast concrete specimens and the beam plates for Specimens P2 and PR2 were ¾ in. (16 mm) thick. The large steel beam angles used in Specimens P1 and PR1 were 5 x ¾ x 14 in. (127 x 10 x 356 mm) long. All welding was done using E7018 electrodes.

**TEST RESULTS**

**Specimen MR**

This specimen was constructed monolithically and was designed to have the plastic hinge develop away from the column face. At 1 in. (25 mm) displacement, a flexural crack of about 0.2 in. (5 mm) in width formed on the tension side of the beam. This crack was located at about 30 in. (760 mm) from the column face. It also indicated that the plastic hinge was beginning to develop at about one-and-a-half times the beam depth from the column face. At a displacement of -2 in. (-51 mm), additional tension cracks formed on the tension face of the beam. The largest of these cracks was 0.25 in. (6 mm) wide. In addition, the tension cracks in the column within the joint region were becoming more noticeable.

After two 2 in. (51 mm) cycles, the specimen was severely cracked, as can be seen in Fig. 7. In the following two 3 in. (76 mm) cycles, further cracking of the beams and columns was observed. Load vs. displacement for Specimen MR is plotted in Fig. 8. This plot does not show the second 3 in. (76 mm) cycle because the test was interrupted due to equipment problems and the hysteresis response of the last cycle was not plotted.

**Specimen P1**

Specimen P1 was constructed using precast concrete members with the
beam plastic hinge expected to form at the column face. Several flexural cracks were observed after the first 1 in. (25 mm) cycle. At the end of the second 2 in. (51 mm) cycle, the cracking was very extensive (as shown in Fig. 9) and the column plates were beginning to separate from the concrete. During the first 4 in. (102 mm) cycle, a loud popping sound was heard as one of the beam bars broke and the test was terminated shortly following fracture of the bar. This bar broke prematurely, probably due to the brittleness introduced by the welding of the bar to the plate. A plot of load vs. displacement for Specimen P1 is shown in Fig. 10.

**Specimen PR1**

Specimen PR1, fabricated using precast concrete components, was designed to relocate the plastic hinge away from the face of the column.

After the first 1 in. (25 mm) cycle, there were several small cracks within the beam-to-column joint. As the test proceeded through the two 2 in. (51 mm) cycles, cracking was observed throughout the beam and column. There were also noticeable cracks forming around the column plate and beam angles. The most severe cracking was at the plastic hinge region, near 1.5d, where the cracks were up to 0.20 in. (5 mm) wide. The cracks within the joint, for example, were only about 0.01 in. (0.25 mm) wide.

During the first 3 in. (76 mm) cycle, a large tensile crack formed near the steel angle on the beam. After the second 3 in. (76 mm) cycle, the plastic hinge was well defined as can be seen in Fig. 11, and the crack near the beam angle was over 0.25 in. (6 mm) wide. During the first 4 in. (102 mm) cycle, at a displacement of 3½ in. (89 mm), a loud popping sound was heard. This was due to the breaking of one of the reinforcing bars in the beam near the face of the column, and the test was ended. A plot of load vs. displacement for Specimen PR1 is shown in Fig. 12.

**Specimen P2**

As explained earlier, the final two precast concrete specimens incorporated a different connection detail in addition to longitudinal No. 6 beam bars, as compared to the No. 7 bars used in the previous three specimens. Specimen P2 was designed without any intermediate reinforcement.

During the first 1 in. (25 mm) displacement cycle, two cracks developed, both emanating from the top of the angle embedded in the beam. One
crack extended from the top of the angle at approximately 45 degrees to the column, while the other moved from the top of the angle into the beam along the longitudinal beam bars. These cracks continued to widen through subsequent cycles until the eventual failure of one of the welds along the side of the plate.

Following two cycles at 1½ in. (38 mm), an attempt was made to displace the specimen to 2 in. (51 mm), but at a displacement of 1.79 in. (45 mm) one of the beam bars on the plate side of the beam fractured. It was decided to conclude the test at this point due to the violent nature of the failure. Fig. 15 shows extensive cracking around the beam plate following the test. The load vs. displacement plot for Specimen PR2 is shown in Fig. 16.

### COMPARISON OF TEST RESULTS

#### Moment Capacity

One measure of performance of a test specimen is to compare its measured flexural capacity with the nominal moment capacity based on the actual measured material properties. The test is considered successful, at least as far as overall moment capacity is concerned, if the specimen is able to resist this design load. The ratio of the maximum measured moment to the nominal moment capacity for Specimens MR, P1, PR1, P2 and PR2 are 1.31, 0.98, 1.04, 1.07 and 1.20, respectively. All tested specimens, except Specimen P1, resisted moments which were larger than their nominal moment capacities.

#### Energy Dissipation

A measure of the energy dissipated by a specimen is represented by the area enclosed within the load vs. displacement curves for any given load cycle. The energy dissipation is a measure of the ductility of the specimen. The energy dissipated at each cycle is normalized with respect to the product \( P \Delta y \). This normalization is needed to...
eliminate the effect of the members having different amounts of flexural reinforcement. The normalized amount of energy dissipated for each load cycle is shown in Table 2. Table 2 shows that the energy dissipated by each of the two precast Specimens P1 and PR1 was higher than that of the monolithic Specimen MR. Although Specimen MR resisted more load, Specimens P1 and PR1 were able to dissipate more energy. Specimens P2 and PR2 did not dissipate as much energy as Specimen MR. This is because of the excessive cracking and rapid deterioration of the concrete surrounding the beam seat plate.

### Strain Gauge Data

As mentioned earlier, strain gauges were attached to the steel reinforcement at locations of interest. The strains in the main beam bars are shown in Fig. 17. The gauge for Specimen PR2 did not work and, thus, there is no plot for this specimen. The plots are very similar and, as expected, the strains in Specimens MR and PR1 were less than those of Specimen P1. This is due to the presence of intermediate reinforcement in Specimens MR and PR1.

The strains in the beam stirrups closest to the column are shown in Fig. 18. It was interesting to note that the strains in Specimen MR were only a fraction of the strain in all the precast concrete specimens. Specimen P1 had much higher strains than other specimens. This can be expected because Specimen P1 did not have the plastic hinge relocated away from the column face and the stirrups, which had strain gauges attached to them, were located near the column face where the plastic hinge developed. Specimens P2 and PR2 did not show the same response because the tests of these specimens were stopped before the 3 in. (76 mm) cycles, which is when extensive cracking usually occurred in the beams.

The plots of the strain in the intermediate beam bars are shown in Fig. 19. Comparing Specimens MR and PR1, it was concluded that the slightly higher strains in Specimen MR were due to the continuity across the beam-to-column interface in that specimen. After reviewing the data from Specimen PR2, it was shown that by continuing the intermediate reinforcing bars from the tension side into the compression side of the beam, the intermediate bars were able to develop more resistance than was expected from the results of the other two tests. This means that the connection detail used in Specimen PR2 (see Fig. 4) proved to be a better way to anchor the intermediate beam bars.

The plots of the strain in the column bars are shown in Fig. 20. Although the plots were similar, the monolithic specimen, MR, resisted more strain than the precast concrete specimens. This was consistent with the load vs. displacement plots which showed that for the same amount of deflection, the monolithic specimen (MR) resisted slightly more load than the precast concrete specimens (Specimens PR1 and P1). For Specimens P2 and PR2, the damage was concentrated in the...
beams and, consequently, the strains in the column bars remained well within the elastic range.

The plots of the strain in the column hoops are shown in Fig. 21. The strains were similar for all specimens. Specimen P1 consistently had slightly higher strains, but that is because it resisted more load than the relocated plastic hinge Specimens MR and PR1. For the same reason, Specimen P2 had more strain on the column hoops than Specimen PR2 as the test proceeded past the second 2 in. (51 mm) cycle. The data clearly indicate that joint reinforcement similar to that suggested for monolithic construction is adequate for precast connections.

Table 3. Joint shear deformation.

<table>
<thead>
<tr>
<th>Displacement (in.)</th>
<th>MR</th>
<th>PR1</th>
<th>P1</th>
<th>P2</th>
<th>PR2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.16</td>
<td>1.80</td>
<td>0.95</td>
<td>2.04</td>
<td>1.40</td>
</tr>
<tr>
<td>2</td>
<td>5.14</td>
<td>4.85</td>
<td>2.95</td>
<td>3.76</td>
<td>2.18*</td>
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<tr>
<td>3</td>
<td>7.60</td>
<td>5.99</td>
<td>5.35</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

* At maximum displacement of 1.66 in. (42 mm).

Displacement Transducer Data

For each specimen, eight displacement transducers were mounted on the beam-to-column connection and on the sides of the beam as shown in Fig. 6. The data from these sensors were used to evaluate the joint shear deformations for each specimen. The total joint shear deformation is defined as the sum of the average vertical and average horizontal components. Fig. 22 shows the deformed joint configuration. Table 3 gives the total joint shear deformations at recorded displacements. The higher the total joint shear deformation, the more cracks are formed in the beam-to-column connection.

It can be observed that the concentration of damage in the joints of the
precast concrete members (Specimens PR1 and P1) was substantially less (21 and 30 percent, respectively) than the similarly constructed monolithic specimen (MR). In other words, most of the deformation in the precast concrete frames, especially at lower load levels, was concentrated in the beams away from the columns, resulting in smaller joint deformations.

Crack Patterns and Failure Modes

Throughout the tests, crack development was observed, marked and photographed at regular intervals and at points of interest to provide an accurate record of crack development. The crack patterns are shown in several figures. These patterns show that the details used were effective in removing the plastic hinges from the faces of the columns for Specimens MR, PR1 and PR2. The cracking within the beam-to-column connections and within the columns for each specimen appear similar, but the displacement transducers showed the differences previously discussed.

The eventual failure of each precast concrete specimen was caused by the fracture of one of the beam bars. This indicates a localized problem near the beam plates or angle. Fig. 23 shows this crack for Specimen PR1; Fig. 13 shows this crack for Specimen P2. For both details, the steel plate or angle rotated excessively. This, in turn, is believed to have induced additional stresses to the beam bars, which when added to the tensile stresses from the beam bending action, could have contributed to the specimens failing earlier than desired. In addition, the beam bars may have become brittle due to their welding to the steel angles or plates, which may have contributed to the undesirable failure modes experienced.

CONCLUSIONS

The two areas studied in this experimental program were the relocation of plastic hinging zones and the ductility of moment-resisting precast concrete connections. The research objectives were to establish plastic hinges away from the column face and to develop moment resistant connections which would perform similarly to monolithically cast connections. The overall performance of the test specimens was found to be satisfactory. The following conclusions are drawn from the results of this investigation:

1. The beam plastic hinging zone can be relocated away from the column face by using intermediate layers of longitudinal reinforcement and some tension and compression bars at a predetermined location in a beam. This was achieved for both the monolithic and precast concrete specimens.

2. The precast concrete specimens with the relocated plastic hinges were comparable in strength and ductility to cast-in-place specimens where plastic hinges usually form near the column face. The relocation of plastic hinges caused cracking in the beam region, which is a very desirable characteristic, especially for seismic loading.

3. The precast concrete specimens with specific connection details used in this experimental program performed similarly to that of monolithically cast concrete connections. The precast concrete connections were strong enough to force the formation of a plastic hinge away from the column face.

4. The critical part of the precast connections was the welded beam bars as they initiated the failure of the specimens. Special attention must be paid to the quality of welding at the precast plates and reinforcing bars.

5. It was observed that the strains in the beam stirrups near the column faces of the precast concrete members were much greater than that of the monolithic member. Hence, because of the discontinuity of the precast concrete members, additional shear reinforcement is necessary near the beam-to-column connections in precast concrete frames.

6. The shear forces within the connection region of the precast and the monolithic specimens were almost identical. It is concluded that the confinement reinforcement used within the joint for monolithic members is adequate for precast concrete members.

7. The intermediate reinforcing bars in the precast concrete specimens had less effect on the capacity of the specimen early in the test; however, as the test progressed, these bars contributed to the specimen capacity.

8. The recommendations of ACI-ASCE Committee 352 for limiting joint shear stress and providing confinement for the joint region appear to
be adequate when applied to precast concrete frames.

RECOMMENDATIONS FOR FURTHER RESEARCH

The results of this experimental program answered many questions about precast concrete connections and the relocation of plastic hinges. Some recommendations exist for the design of these specimens, but to develop specifications, additional testing must be performed on precast concrete connections with and without relocated plastic hinges.

Future designs of precast concrete connections should be performed in such a way as to account for columns being slightly out of plumb or the beams being slightly longer or shorter than specified, as was done on Specimens P2 and PR2. The ability to use connections which allow for these imperfections would make them more practical and realistic for actual field construction.

The specimens with the relocated plastic hinges had much less cracking within the connection region. It may be possible to relax the reinforcement requirements within the joints if the relocated plastic hinge concept is used. Further testing could determine how these requirements may be reduced.

The use of angles or plates caused the corners of the beam (where connected to the column) to break away. A possible solution would be to connect the angles which are welded to the beam's tension and compression reinforcement together with steel bars or plates to help resist or reduce cracking. The effectiveness of this and other similar approaches must be investigated.

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