PCISFRAD Projects 1 and 4

Summary Paper

Moment Resistant Connections and Simple Connections

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CONTENTS

Note: This paper is a condensation of PCI-SFRAD Project No. 1/4, Moment Resistant Connections and Simple Connections. The full report is available from PCI Headquarters at $20.00 to firms supporting the sponsored research, $30.00 to PCI Members (non-supporting firms), and $60.00 to non-PCI Members.

The summary paper, and the full report, are based on a research project supported by the PCI Specially Funded Research and Development (PCISFRAD) Program. The conduct of the research and the preparation of the final reports for each of the PCISFRAD projects were performed under the general guidance and direction of selected industry Steering Committees. However, it should be recognized that the research conclusions and recommendations are those of the researchers. The results of the research are made available to producers, engineers and others to use with appropriate engineering judgment similar to that applied to any new technical information.

The successful structural performance of precast concrete systems depends upon the connection behavior. The configuration of the connections affects the constructibility, stability, strength, flexibility and residual forces in the structure. Furthermore, connections play an important role in the dissipation of energy and redistribution of loads as the structure is loaded.

The PCI Specially Funded Research and Development Programs 1 and 4 (PCI 1/4) focused on the actual behavior of commonly used connections. The two programs were combined in order to devote maximum effort to the physical testing of connections in common use. PCI 1/4 consisted of individual tests of eight simple connections, eight moment resisting connections and one moment resisting frame test.

This condensed report summarizes the test program, describes the test specimens, and presents the basic findings and conclusions reached during the investigation. The Research Report[1] contains a complete description of the research program, as well as detailed descriptions of the individual tests.

1. RESEARCH OBJECTIVES AND BACKGROUND

Simple connections are designed to resist or transmit shear and axial loads. The Project Steering Committee selected the simple connections for this program. The study team reviewed the prevailing design approaches for each connection, formulated a design capacity based on the nominal specified material strengths, and predicted capacities based on the actual measured material properties. One test was performed on each connection. The results of the tests were compared to the predictions and conclusions were drawn.

Moment resisting connections are essential to develop frame action in precast buildings. The connections must develop sufficient strength to resist the applied loads and must have sufficient stiffness to limit the sidesway of the structure. The Project Steering Committee selected the moment resisting connections for investigation based on the experience of the members.

In this study, connections were examined for structural performance, as measured by load and deflection behavior, and for cost effectiveness and constructibility. Emphasis was placed on the behavior of the connection subjected to gravity loading and lateral loading due to wind or equivalent seismic loading. Although rigorous seismic analyses were not performed, several of the connections were subjected to moderate cyclic loadings in order to observe their behavior.

A two-bay by two-story moment resisting frame was constructed. The frame included several moment resisting connections similar to those tested individually. The frame was tested as a general confirmation of the findings of the individual tests and offered an opportunity to refine some of the details and recommendations derived from the individual tests.

The moment resisting frame design methodology used to define the test loadings and some of the connection detailing included a review of the recommendations of the 1985 Uniform Building Code (UBC).[2] This design approach was influenced by the consideration that the implementation of moment resisting connections may be subjected to a building department review and that UBC is a widely recognized acceptance standard.

Many structures subjected to lateral loadings from wind or seismic forces are designed to resist equivalent static loads. Analyses are performed using linear elastic methods and the resulting effects are factored to obtain strength design values. The equivalent static loads are specified in codes, such as UBC, and are predicated on the assumption that the structure will possess an assumed level of ductility roughly equivalent to that of a well detailed monolithic structure.

Test specimens used in this program were not detailed for the specific confinement criteria of UBC Seismic Zone 3 as it was felt that many of the connections would be used for construction in Zones 1 and 2. Thus, these test results would be applicable for these locations. Consequently, some specimens failed at loads slightly lower than those which might have been reached if details such as closer stirrup spacing and 135 degree hooks were used.

2. MOMENT RESISTING CONNECTIONS— DESCRIPTION AND FINDINGS

Eight basic moment resisting connections were studied. They are defined in the following paragraphs and summarized in Fig. 1. The test results are summarized in Table 1. Tabular results are given for Specimens BC28 and BC29 even though they represent the same basic connection type.

Connection BC-15 is a classical welded plate connection. In this connection, a steel plate is welded to the principal longitudinal reinforcement. A companion plate is cast into the adjacent precast member, in this case a column element. A closure plate is field welded between the two embedded plates to complete the connection.

Under negative moment, the connection failed when one of the bars fractured at the end of the weld and when the field welds failed. Failure was a result of direct tension and the prying action of the beam end rotating about the outer edge of the column corbel (see Fig. 2). Detailed designs which reduce the eccentricity of the load path through the connection and which minimize the prying due to the corbel should further improve the connection performance.

The positive moment capacity of the connection reached a value comparable to the negative moment design value. The connection displayed limited energy dissipation under cyclic loading.

Connection BC16A obtains its moment capacity by using continuous reinforcement cast into a composite topping. The moment resistance of this connection was considerably greater than predicted. In addition, the connection displayed considerable ductility. While the connection displayed strength and ductility, the energy dissipation during cyclic loading was low. The strength of the connection was adversely affected by the prying action of the beam rotating about the corbel.

Connections BC25 and CC1 are beam-to-column and column-to-column connections. Each connection is made by bolts between end plates cast into the column elements. Two different connection configurations were tested. Specimen BC25 was loaded with a 300 kip (1330×10^3 N) axial load and high strength bolts provided moment resistance. Specimen CC1 was loaded with a 40 kip (178×10^3 N) axial load and used ASTM A-307 anchor bolts.

Both specimens were loaded cyclicly. Specimen CC1 did not fail after three full cycles. Specimen BC25 failed on the third cycle by bursting of the ties and buckling of the reinforcement. Both connections exceeded their predicted capacity and, in the case of Specimen BC25, by a factor of nearly two. Neither connection dissipated much energy

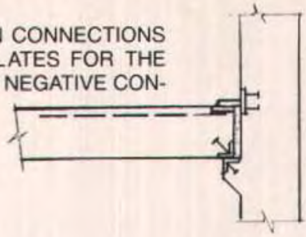
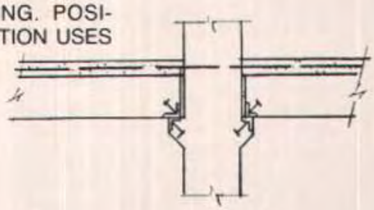
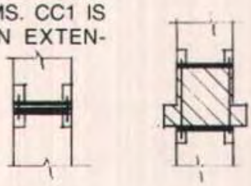
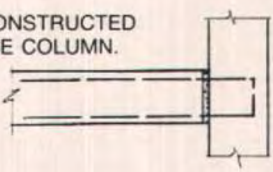
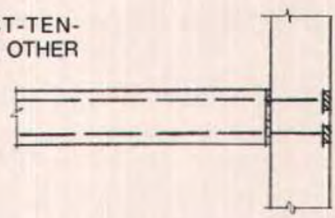
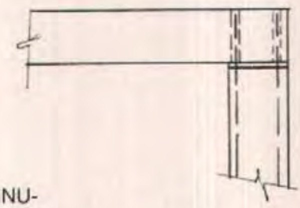
CONNECTION	DESCRIPTION
BC15	BEAM TO COLUMN CONNECTIONS USING WELDED PLATES FOR THE POSITIVE AND THE NEGATIVE CONNECTIONS. 
BC16A	BEAM TO COLUMN CONNECTION USING CONTINUOUS REINFORCING THROUGH THE COLUMN AND CAST-IN-PLACE TOPPING. POSITIVE MOMENT CONNECTION USES WELDED PLATES. 
BC25 & CC1	A COLUMN BASE DETAIL MADE CONTINUOUS BY BOLTING. BC25 IS FOR EXTENDING COLUMNS THROUGH LARGE BEAMS. CC1 IS A COLUMN-TO-COLUMN EXTENSION. 
BC26	A PRECAST BEAM CONSTRUCTED INTO A CAST-IN-PLACE COLUMN. 
BC27	A PRECAST BEAM POST-TENSIONED TO A COLUMN OR OTHER FRAME MEMBER. 

Fig. 1. Moment connections.

CONNECTION	DESCRIPTION
BC28 & BC29	A PRECAST BEAM INSTALLED ON A GROUTED OR PARTIALLY GROUTED DOWEL.



BC99	A PRECAST BEAM MADE CONTINUOUS USING DYWIDAG THREAD BARS SCREWED INTO COUPLERS CAST IN THE COLUMN.

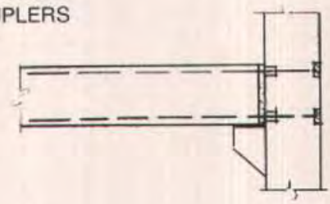


Fig. 1 (cont.). Moment connections.

during cyclic loading.

The bolt capacity limited the strength of the connection and bolt yielding accounted for almost all of the energy dissipation. Specimen BC25 could develop the full bolt capacity if additional tie confinement was used. The sudden buckling failure of the bars and the possible implications for frame collapse suggest that this connection may not be reliable as the principal moment resisting element. A connection located closer to the point of inflection, rather than the point of maximum moment, may be more appropriate and 135 degree hooks should be used on ties near the connection plates.

The bolts in Specimen CC1 deformed plastically during the first loading. Subsequent load cycles displayed little moment resistance until the anchor plates regained contact with the elongated bolts.

Connection BC26 simulates a cast-in-place connection. It is awkward to construct since it requires shoring, yet its similarity to cast-in-place construction makes it the logical benchmark for strength and ductility comparisons.

The specimen was tested monotonically. Its strength and ductility exceeded predictions. The specimen failed when shear ties pulled out. In this case, stirrup confinement using 135 degree anchorage provisions could have added capacity and ductility to the connection. The fact that the flexural strength at failure exceeded the predicted strength is due to strain hardening of the reinforcement. When strain hardening effects are included, the predicted strength agrees well with the ultimate strength.

Connection BC27 is a fully post-tensioned connection. The specimen was post-tensioned with strand in each corner. The specimen displayed good ductility and the strength slightly exceeded the predictions. Failure occurred by fracture of one of the strands.

Table 1. Results of moment resisting connection tests.

Connection	Negative moment			Positive moment	Maximum rotation	
	Design moment (kip-in.)	Predicted capacity (kip-in.)	Measured capacity (kip-in.)	Measured capacity (kip-in.)	Negative (percent)	Positive (percent)
BC15	1428	1904	2185	1450	3.7	1.20
BC16A	1428	1904	3500	1218	10.0	4.00
BC25	1587	2488	4535	4228	4.0	4.00
CC1	1020	1632	1523	1575	4.0	3.65
BC26	1428	1904	3100	No test	12.5	No test
BC27	2086	2575	2388	No test	8.3	No test
BC28*	291	399	576	No test	4.1	No test
BC29*	291	399	540	318	3.9	3.75
BC99	1428	1904	2186	768	4.0	2.90

* The self weight of the beam is 164 kip-in. or 51 percent of the reported capacity.
Metric (SI) conversion factor: 1 kip-in. = 113 N•m.

Post-tensioning provided good initial stiffness, equivalent to that of uncracked concrete with a modulus of elasticity of 7400 ksi (51,000 MPa). This stiffness remained effective until the initial post-tensioning force was overcome.

Connections BC28 and BC29 are dowel connections which are designed to eliminate field welding. The total moment capacity of these connections was comparable to the design value. The low magnitude of the achievable moment and the large deformations required to develop this moment makes them unlikely choices as principal sources of moment resistance for frame action. They do provide some resistance and may be useful in some situations.

Fig. 2. Connection BC15.

Connection BC99 is designed to obtain full moment resisting capacity by using mechanical couplers in lieu of field welding. A cast-in-place closure is required to complete the connection. Field reports and experience building the test specimen indicated that it can be constructed quite easily.

The test specimen failed by sudden rupture of the connectors at approximately the predicted capacity (see Fig. 3). Investigation of the specimen determined that the couplers were manufactured to develop only the specified yield strength of the bars. Thus, the couplers had no reserve capacity nor did they display any ductility. The coupler manufacturer makes a coupler which can

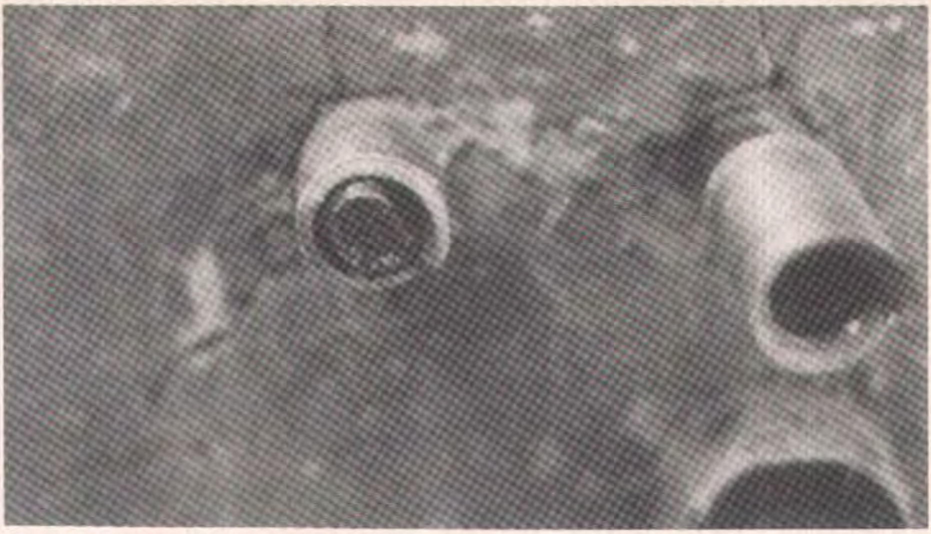


Fig. 3. Specimen BC99 — top couplers at failure.

develop the ultimate capacity of the bar. The high strength coupler was used in the frame test of this connection and the connection behaved well.

The availability of two similar couplers indicates a potential quality control problem. The sudden loss of ductility with the yield strength couplers makes them undesirable for use in moment resisting frames.

3. SIMPLE CONNECTIONS — DESCRIPTION AND FINDINGS

The eight simple connections are described in the following paragraphs and are shown in Fig. 4. A summary of the design, predicted and measured capacity, is given in Table 2. Where appropriate, the recommendations of the PCI Design Handbook[3] were used to design and detail the connections. Thus, some verification of the Design Handbook was obtained.

Connections SB1 and SB2 are designed as the top and bottom connections to resist end torsion in a spandrel beam. The estimated force in these connections is 5 to 15 kips (22×10^3 to 67×10^3 N) based on a range of assumptions regarding the length of the spandrel beam and the eccentricity of the beam bearings. The connections consist of plates or angles cast into the precast elements and a closure plate or angle which is field welded to the embedded plates.

Both connections performed in excess of the design requirements. The use of an angle for the closure plate proved to be resilient and appeared to be less sensitive to construction tolerances.

Connection CB2 consists of a series of tests on coil rod inserts. The pullout resistance of the five tests varied considerably and the pullout strengths were generally less than the capacities predicted using design methods outlined in the third edition of the PCI Design Handbook. (Note that the second edition of the PCI Design Handbook tends to overestimate the strength of these

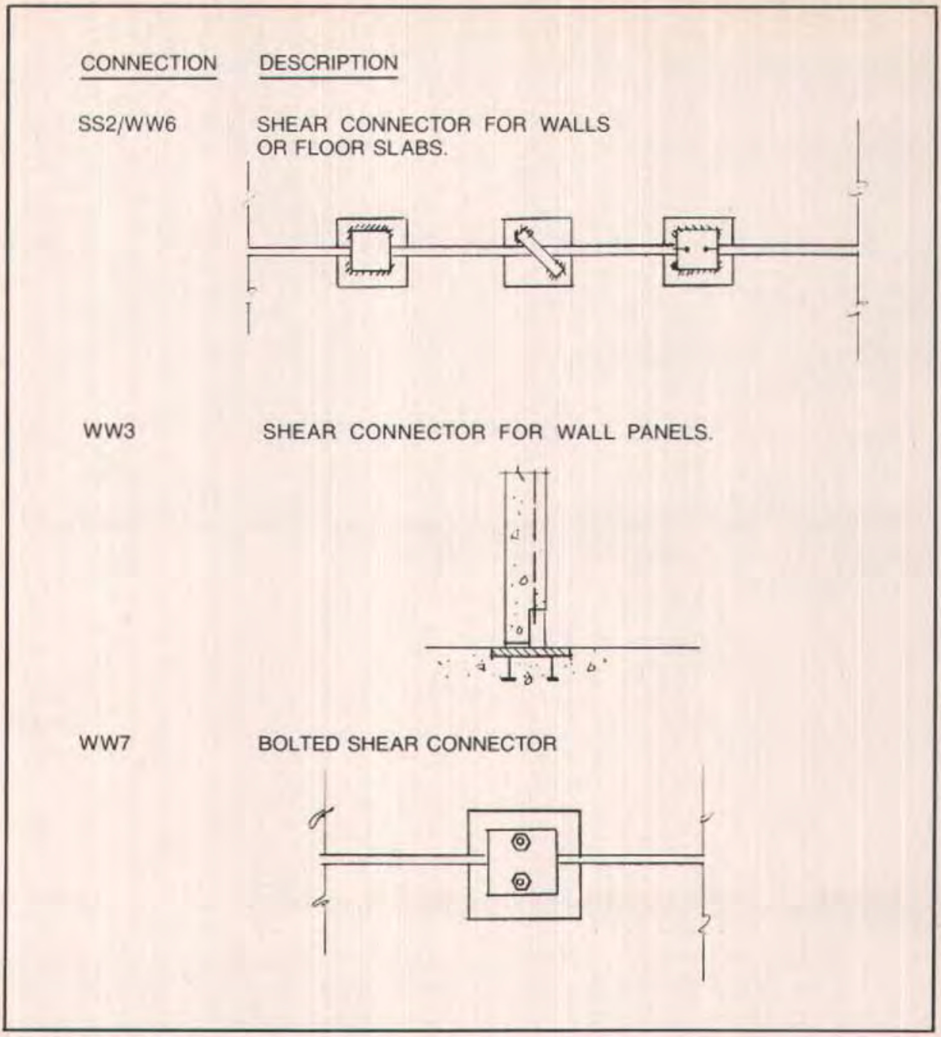


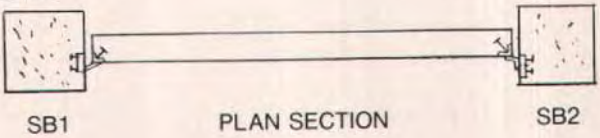
Fig. 4. Simple connections.

connections.) The lower strengths may have been partly due to bending in the test panel, however, these inserts are often used in such panels. The tests supported the manufacturers' recommendations that factors of safety of four against pullout (ultimate strength) are entirely appropriate.

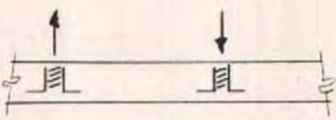
Connection CB3 is an embedded shear connector designed to resist gravity or other in-plane shearing loads. The connector consists of reinforcement welded to an angle which is cast into a thin panel. The connector exhibited good ductility and strength. Failure occurred by fracture of one of the welded bars and was preceded by extensive cracking of the panel.

Connections SS2, WW6, WW7 are in-plane shear connectors. The test specimen consisted of connector plates field welded or bolted to inserts cast into the precast panels. All of the welded connectors exceeded the

CONNECTION	DESCRIPTION
SB1	WELDED PLATE CONNECTION TO RESIST TWISTING OF A SPANDREL BEAM.
SB2	WELDED ANGLE CONNECTION TO RESIST TWISTING OF A SPANDREL BEAM.



CB2	COIL ROD INSERT



CB3	THIN WALL PANEL GRAVITY LOAD CONNECTOR

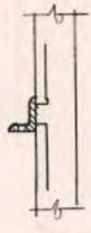


Fig. 4 (cont.). Simple connections.

strength requirements for the connection. The bolted connections were unable to develop design strengths.

Several plate configurations were examined to determine if the ductility of the connector could be improved. It is desirable to have the failure occur in the connector plate material rather than in the weld. At the same time, it is sometimes desirable to reduce the stiffness of the connection perpendicular to the shear plane in order to limit the forces due to shrinkage and thermal effects. Of the various alternatives, a flat bar inclined to the shear plane exhibited substantial promise.

Connection WW3 is an angle assembly used to erect precast panels and to provide shear resistance across a horizontal joint between the panel and its foundation. The connection proved to be quite ductile, tough and predictable. Even after the failure of some of the studs it was able to retain a significant

Table 2. Results of simple connection tests.

Connection	Design capacity (kips)	Predicted capacity (kips)	Measured capacity (kips)	Ductile
SB1	5.0-15.0*		20.6	No
SB2	5.0-15.0*		58.8	Yes, did not fail
CB2t	5.0	10.5	9.5†	No
CB2c	6.0	17.5	12.7†	No
CB3	9.0	24.0	30.8	Yes
SS2	10.0-20.0*	22.0-27.0	31.9†	No, solid plates
WW6	10.0-20.0*	22.0-27.0	31.9†	Yes, stress relieved
WW3	10.0-20.0*	48.0	50.1	Yes
WW7b	7.6-12.5		3.0	Yes, slipped on bolts
WW7w	23.8	28.0	44.0	Yes, when welded

* Nominal design range, no specific design value was developed.
† Average of test results reported.
Metric (SI) conversion factor: 1 kip = 4448 N.

residual load. Because the studs lie outside the panel reinforcement, the use on long studs is vital to the connection integrity.

4. FRAME TEST

A two-story, two-bay test frame was constructed using several different moment resisting connections. The negative moment connections all performed well and the frame was able to sustain three cycles of lateral drift of ±4 percent. The frame was loaded to a lateral deformation of 8 percent of the total story height before experiencing a significant loss of lateral resistance. Several positive moment connections failed as the drift exceeded 4 percent.

5. CONCLUSIONS AND RECOMMENDATIONS

Conclusions and recommendations are given for moment resisting connections and simple connections.

Moment Resisting Connections

Connections BC15, BC16A, BC25, CC1, BC26, BC27 and BC99 all developed strengths at least equal to the predicted values based on their nominal strengths and could be considered strong enough for their intended use. The available ductility varied considerably among specimens. Only Connection BC-26 exhibited as much energy dissipation as would have been expected from a well detailed monolithic joint. Several connections indicated that redesign would improve the ductility performance.

All connections which rely on corbels as part of their erection or load carrying systems will be subjected to prying actions during large deformations. Designs which minimize the leverage created by the corbel should behave better.

The tension load path through a number of connections (such as Connection BC15 were intricate, leading to important deformations which are not normally considered in the design process. Connections which minimize the load path eccentricities should provide better and more predictable behavior. The number of times that the load is transferred within the connection should also be minimized, since each transfer introduces additional possibilities for a failure.

Some of the connections contained less positive than negative moment reinforcement, leading to lower flexural strength in the positive direction. While this may be appropriate for gravity loadings, it may not be satisfactory for developing frame action in the presence of reversible cyclic loading. This is especially true for connections which are not precompressed by the addition of the dead loads.

Simple Connections

The intended purposes of the simple connections were quite diverse; hence, evaluation of their effectiveness must be on a case by case basis. Furthermore, the combinations of loads for which the combinations are designed vary with application. Within these constraints all the simple connections, with the exceptions of Connections SS2 and SB2, provided a reasonable level of strength.

Some of the specimens, such as WW3, deformed in ways which were not anticipated, emphasizing the need for thorough testing of standardized connections.

Connections such as WW6 and WW7 may need strength in one direction and flexibility to accommodate shrinkage in the other direction. To achieve this combination of behavior requires very careful design, especially if the placement of the elements of the connection also has to allow for fabrication and erection tolerances. Suggestions to meet these goals are provided and some suggested details were tested.

It is generally not feasible to dissipate enough energy in the connections for them to be considered the sole source of damping under reversible cyclic loading. However, connections which force a significant inelastic deformation into the member, such as CB3, or which provide substantial shear friction resistance, such as WW3, provide a significant potential for energy dissipation.

The coil rod inserts of Specimen CB2 failed at less than the predicted loads. However, further study is needed on the effects of panel proportions and the effects of other stress conditions in the panels.

General

The test results clearly showed the need for imaginative design and detailing, i.e., not sensitive to minor changes in fabrication and erection, and the need for good quality control. Some specific examples are cited below.

The use of weldable reinforcement (ASTM A706) and the appropriate weld materials are essential for ductility. Even when weldable reinforcement is used, load eccentricities should be eliminated where possible. For example, bars in Specimen BC15 fractured in the heat affected zone at the end of the weld with almost no necking. This location was subjected to severe kinking caused by eccentric load paths.

The original BC99 tests failed in a brittle manner because the high strength couplers were not installed. It is recommended that only full strength couplers be used for this connection.

Some weld plate designs may be predicted on the plate yielding before the welds fail. Overstrength plate steel could then result in welds failing first, sometimes in a brittle fashion, when a ductile failure was anticipated.

6. CLOSURE

Since only one connection of each type was tested, it is imprudent to draw further conclusions at this summary level.

The detailed Research Report[1] contains considerably more test data and interpretation of the behavior of the connections. The authors feel that the data is valuable to the design profession, the precast fabricator and to the building official reviewing precast concrete structures.

REFERENCES

1. Stanton, J. F., Anderson, R. G., Dolan, C. W., and McCleary, D. E., "Moment Resistant Connections and Simple Connections," PCI Specially Funded Research and Development Program—Research Project No. 1/4. Prestressed Concrete Institute, Chicago, Illinois, 1986, 436 pp.
2. *Uniform Building Code (1985 Edition)*, International Conference of Building Officials (ICBO), Whittier, California.
3. *PCI Design Handbook – Precast and Prestressed Concrete*, Third Edition, Prestressed Concrete Institute, Chicago, Illinois, 1985.

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NOTE: Discussion of this report is invited. Please submit your comments to PCI Headquarters by December 1, 1987.