A PROPOSED INDUSTRY-STANDARD BEAM CONNECTION

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Examples of connection distress (shear at beam end reactions) are shown in Figs. 1 and 2. Fig. 1 shows a beam-to-column bearing connection with two potential failure locations. The column may split, or the corner may break off. Essentially no reinforcing steel crosses either crack location. Failures like these have caused collapses. Fig. 2 shows a beam-to-column corbel connection with potential failure locations. Again, little or no steel crosses the corner cracks. The crack at the column face may have insufficient reinforcement crossing it, or the reinforcement may be insufficiently anchored on the corbel side of the crack. These also have caused collapses.

Failures like these, although not common, seem to keep repeating

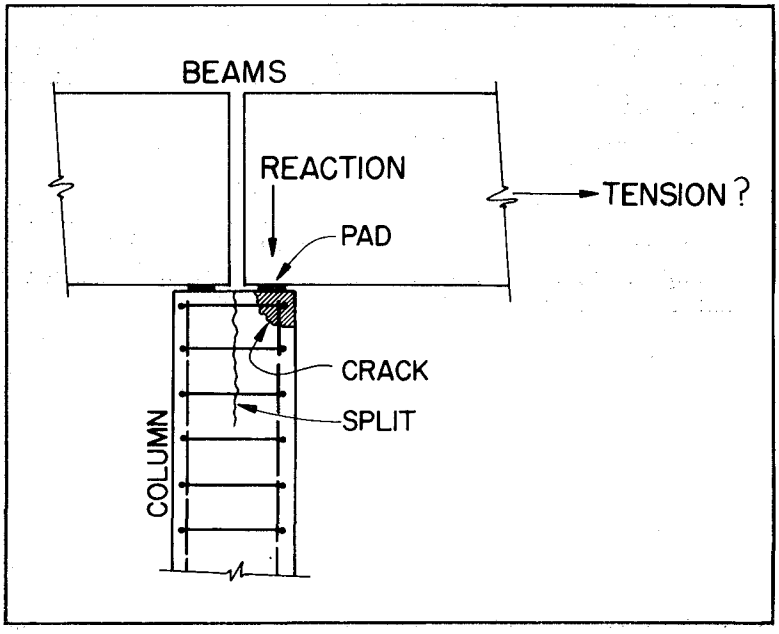


Fig. 1. Potential failure modes in beam-to-column connection

Structural distress in precast concrete connections may be ascribed to lack of good industry-standard connections, and lack of an accepted rational method for designing custom connections. A versatile candidate for an industry-standard beam connection is presented. It is designed by a rational method based on the shear friction theory.

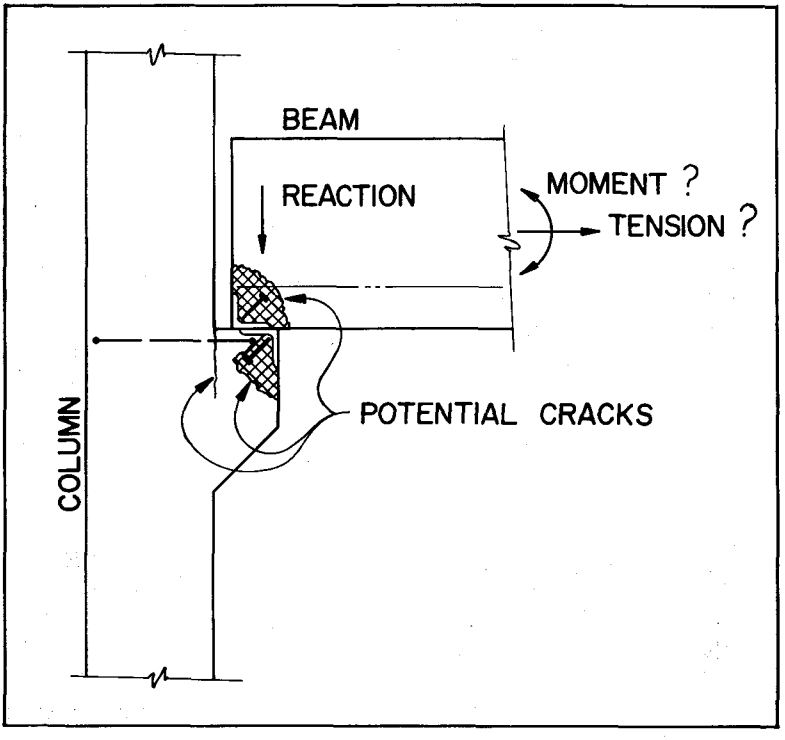


Fig. 2. Potential failure modes in beam-to-column corbel connection

for several reasons:

1. The most highly stressed, and consequently the most critical, part of a precast structure is the connections. This is because major structural loads must funnel through narrow regions. There is no widely accepted, rational method for designing such regions. Consequently, they tend to get detailed but not designed.
2. There are no good, generally accepted standard connection

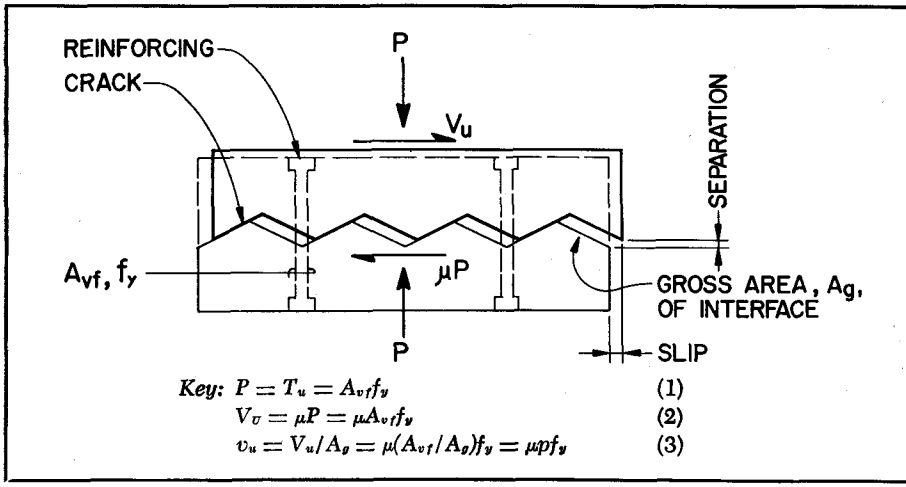


Fig. 3. Shear friction concept

designs or design methods. Designs in cast-in-place concrete or structural steel can take advantage of the enormous wealth of experience on how to avoid trouble contained in the ACI Building Code[4] and in the Steel Design Handbook[5]. These types of structures generally have a lower incidence of trouble in the connections.

3. Members are easy to design, connections are not. Consequently, the design drawings often do not show the connections at all. This isn't surprising, as the designer really has no single source where he can find out how to design them. He may feel the fabricator, being experienced in precast work, knows how but, unfortunately, the fabricator may be in little better position than the designer.

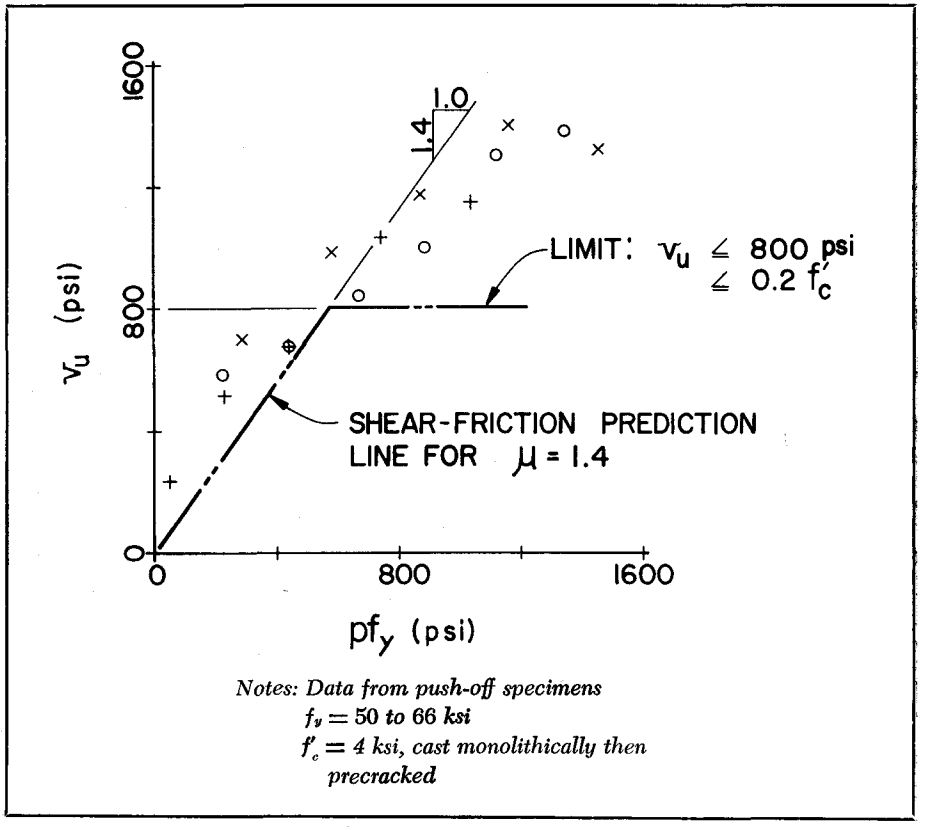
A solution to these problems exists. It is a rational design method for designing connections. It works and it's simple. It's called shear friction. The author's firm has used it successfully for ten years and in some instances for very difficult connections. The main purpose of this paper is to outline this method and to propose a standard beam-to-column connection designed by this method.

SHEAR FRICTION

In precast concrete structures, you will sometimes find cracks in the most unexpected places, and generally in the connection. These occur due to such causes as shrinkage, camber, creep, settlement, erection shocks, and from forcing where pieces don't fit. A good connection design, therefore, will consider the "accidental" crack. It will not depend on the tensile strength of the concrete, but will use reinforcing steel to assure ductility and strength in the presence of the "accidental" crack. Many such cracks involve sliding shear along the crack. Shear friction considers this condition.

Consider shear along the crack shown in Fig. 3. This crack might just as well be a vertical one at the reentrant corner between a corbel and its column face as shown in Fig. 2. The upper block is pushed to the right by the ultimate shearing force V_u. Because the crack is rough, the slippage will cause the two blocks to be pushed apart or separated. However, the two blocks are held together by reinforcing steel of cross-sectional area A_{vf} which must be anchored for full tension on both sides of the crack. The separation forces the steel into tension yield, producing the balancing compressive clamping force P (see Equation 1, Fig. 3). For mild steel reinforcement, a separation of about 0.01 in. is sufficient to develop yield. The compressive force P on the cracked surface then produces a frictional force μP which resists the applied shear V_u (see Equation 2). Here μ is the apparent coefficient of friction, which may be as high as 1.7. It is analogous to tan ϕ, where ϕ is the angle of internal friction used in the analysis of cohesionless soils. If Equation 2 is divided by the area of the shear interface, we get Equation 3, where v_u is the shear stress on the interface at ultimate load and p is the steel ratio.

Many tests show that this shear



Notes: Data from push-off specimens
f_y = 50 to 66 ksi
f'_c = 4 ksi, cast monolithically then precracked

Fig. 4. Shear friction test data

friction concept accurately predicts the lower bound of shear strength. Mattock[1] has run a very careful and thorough series of push-off tests and some of his data is plotted in Fig. 4. The ordinate is the shear stress at ultimate load and the abcissa is the reinforcement index. Note how well the shear friction prediction line forms a lower bound to the data. The limit on v_u of 800 psi is arbitrary; with high steel percentages, usable shear strength goes up to 1200 psi. The tests showed ductile behavior.

The shear friction concept is in the proposed revision to the ACI Building Code[4]. See also Reference 6. The major provisions, with alternate forms of the equations, are given in Table 1.

Equations 5 and 6 are the same as previously developed except for the reduction factor, ϕ. They are valid only in the absence of axial tension (see Reference 2). The limits for v_u and f_y are arbitrary. They are not based on known limits; they are imposed only to prevent extrapolation into what is now unknown.

The shear friction concept has wide application. It is rational. It is easily understood and remembered. It may be safely used for connections having geometry markedly different from those previously tested. It is an extremely useful tool which can keep everybody out of trouble. The main trick is to pick the worst "accidental" crack locations.

A PROPOSED STANDARD BEAM CONNECTION

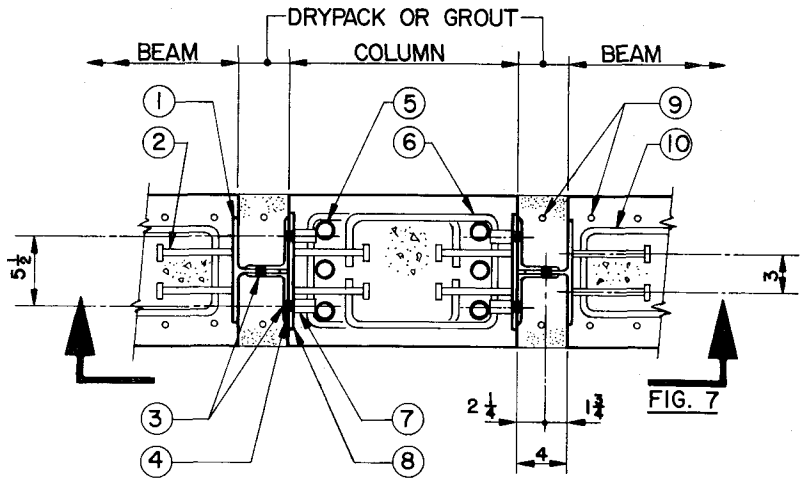
As a standard to compare against, the ideal beam end-connection would have these qualities:

1. It would carry vertical shear independently of moment or horizontal tension. The shear part of the connection would be ductile, and would be able to absorb large rotations and elongations. Many types of precast connections lose their shear capacity when the beam is overloaded in moment or tension. Yet, this shear capacity must remain if safety is to be assured.
2. The connection would be versatile in its application, and would permit both beam-to-girder and beam-to-column connections.

Table 1. Design equations and limitations for shear friction

$v_u \leqq 0.2 f'_c$ or 800 psi, whichever is less
$f_y \leqq 60$ ksi
$\mu = 1.4$ for concrete cast monolithically
$= 1.0$ for concrete cast against hardened concrete
$= 0.7$ for concrete placed against as-rolled structural steel.

$$A_{vf} = V_u/\phi f_y \mu \quad (4)$$
$$V_u = \phi(A_{vf} f_y)\mu \quad (5)$$
$$v_u = \phi(p f_y)\mu \quad (6)$$



Key: 1. *Structural tee, ST 4 WF 15.5 × 1'-6"* (8 × 8 *WF* 31)
2. *Headed studs, ½ in. dia. × 6½ in. long*
3. *Friction bolts, ¾ in. dia. A325.*
4. *Clip angles,* 2—4 × 3½ × 5/16 × 1'-5½"
5. *Column vertical steel*
6. *Column ties, sets of two No. 4 each*
7. *Weld-nuts*
8. *Plate,* 9 × ⅜ × 1'-6"
9. *Stirrups, No. 4*
10. *Hairpins, No. 4*

Fig. 5. Plan view of beam-to-column clip angle connection

3. The connection concept would be amenable to standardization, and would be reasonably economical.

In the steel design manual, the well-known bolted clip angle connection satisfies all of these ideals. The proposed precast connection is similar. It is designed to replace the corbel beam-to-column connection developed in Mast's paper (Reference 2). Mast's connection supported continuous, precast pretensioned 12 x 30-in. beams, spanning 48.5 ft. on 15-ft. centers, composite with a 5-in. cast-in-place slab, and subjected to a 100 psf live load. It was designed to resist a shear of 114.0 kips at ultimate load (or 68.7 kips at service load).

Fig. 5 shows the proposed connection in plan, as a two-sided connection with beams framing into both sides of the column. It could also be designed as a one-sided connection, or as a beam-to-girder connection, without changing the concept. The clip angles and bolts are like a structural steel connection. The weld-nuts are similar to those used in the familiar loop-type screw insert. The studs transfer the shear from the steel parts of the connection into the concrete. The hairpins and ties prevent a failure plane beyond the heads. This connection is essentially "invisible" and looks like cast-in-place concrete.

Fig. 6 shows the connection in elevation, with six rows of bolts. For different loads, it could be anywhere from 3 to 8 rows, without changing the concept.

The connection shown is designed

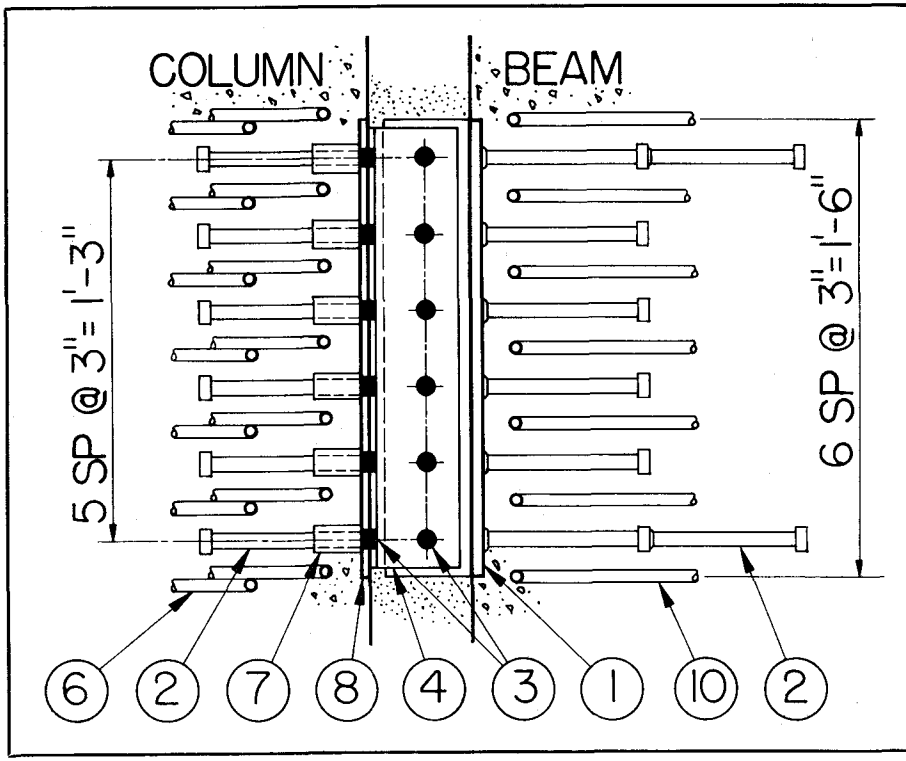


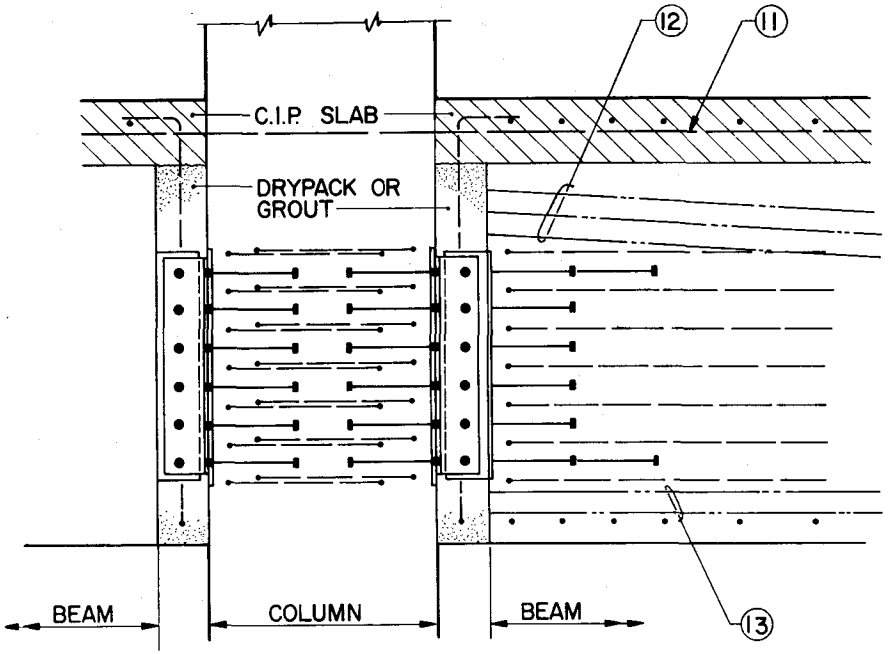
Fig. 6. Elevation of beam-to-column connection (see Fig. 5 for key)

as follows, using working stress design for steel and ultimate strength design for concrete:

1. By shear friction, the maximum interface shear stress, v_u = 800 psi, sets the tee flange dimensions at 8 in. wide by 18 in. high.
2. The angle clips are selected by inspection from the Manual of Steel Construction. The thinnest and most flexible angles, 4 x 3½ x 5/16, are chosen.
3. The tee web should be fully stressed in web shear, to ensure shear yield (and hence ductility) at overload. A structural tee cut from an 8 x 8 WF 31 is chosen.
4. From the steel design manual, ¾-in. diameter A325 friction bolts are selected to obtain small weld-nuts that will not interfere with the column bars.
5. The studs are sized by shear friction. The coefficient of friction, μ, is 0.7 for a steel-to-concrete interface. The yield strength of the studs is 54 ksi (Reference 3). Since the angle clips are flexible in moment and tension, pull-out or tension loads on the studs will be quite small and Equation 5 is valid. However, for safety, the studs are to be long enough (about 10 diameters) to be developed in tension. To obtain better anchorage into the body of the beam, the top and

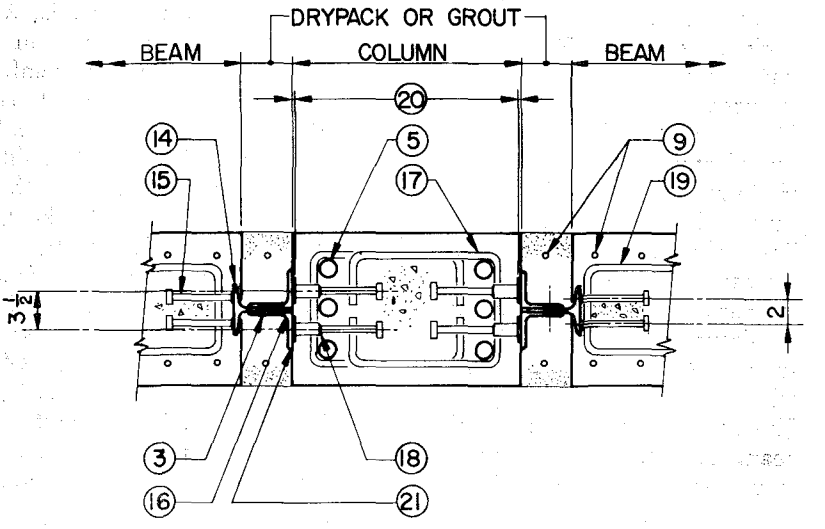
bottom pairs of studs are stacked two high. For an ultimate shear of 114.0 kips, the required cross-sectional area is 3.5 sq. in. Matching the number of rows to bolts, six pairs of ⅝-in. diameter by 6½-in. long studs are selected.

6. The column plate must have the same interface area as the tee flange. Its thickness should be about one-half the stud diameter. A plate ⅜-in. thick is selected.
7. The hairpins in the end of the beam have three functions: 1) to prevent a transverse failure plane at the stud heads; 2) to transfer the end reaction into the beam's principal reinforcement (here, the straight strand); 3) to provide confinement, which prevents splitting in a vertical plane. For the ultimate shear of 114.0 kips, the bar size is chosen by shear friction, using $\mu = 1.4$ (monolithic concrete) and $f_y = 40$ ksi (Grade 40). The required cross-sectional area is 2.4 sq. in. The length is limited by the strand transfer length (50 strand diameters or 25 in.), rather than by the tension an-



Key: 11. *Beam negative moment reinforcing bars*
12. *Beam harped pretensioning strands*
13. *Beam straight pretensioning strands*

Fig. 7. Elevation of beam-to-column connection, complete assembly



Key: 14. *Structural tee, ST 4 I 9.2 (8 × 4 I 18.4)*
15. *Headed studs, ½ in. dia. × 5 in. long*
16. *Clip angles, 2–3½ × 3 × ¼*
17. *Column ties, sets of two No. 3 (Grade 60) each*
18. *Studs welded to weld-nuts*
19. *Hairpins, No. 3 (Grade 60)*
20. *Weld-nuts held back by washers or FB fixing strip, removed after stripping*
21. *Both faying surfaces sandblasted, coated with epoxy or grout, then bolted*

Note: Do not use this concept without extensive testing.

Fig. 8. Plan view of beam-to-column clip angle connection (future state of the art)

chorage length (36 bar diameters or 18 in.). Seven No. 4 hairpins are provided.

8. The column ties are sized like the hairpins. They are shown as hooked stirrups to provide full tension anchorage beyond the adjacent column corner bars. Note that the usual tie cannot provide tensile continuity around the hooked corner.

Fig. 7 shows the connection in elevation. Note that the connection hardware fits well in the end of the beam.

Concerning fabrication and erection tolerances, the major problem is that the bolt holes must match up at erection. This can be handled by holding the hardware by substantial jigs (not by simply wiring to the form), tilting the beam hardware to compensate for camber at release, controlling beam length by locating the beam hardware directly from bolt centerline at one end to centerline at other end (rather than locating relative to end bulkheads), and sub-punching the holes at one end (using sub-sized erection bolts and field reaming for final bolts). These and similar procedures have proved practical on past jobs dating back to 1961.

This particular clip angle connection has not been tested or used in an actual structure, but similar con-

nections have been tested and used successfully. It may, possibly, be too conservative. A future clip angle connection might look like that shown in Fig. 8. This concept has some intriguing possibilities. It is over-stressed when analyzed by the proposed revision to ACI 318. However, any code provision must be conservative enough to be safe for all conditions. Limited test data indicate that ACI 318 is somewhat too conservative for this particular condition. Further testing would probably result in minor modifications, but the concept still applies.

CONCLUSIONS AND RECOMMENDATIONS

1. Connections are the most critical part of the structural concept for precast concrete, and are the cause of most cases of structural distress. Until good standardized connections have become commonplace, connections should be detailed by the structural designer, as he is the one most familiar with the structural concept and is best able to guard against failure due to secondary effects.
2. Shear friction is a simple and powerful design tool for design of connections and its use is highly recommended.
3. An industry-standard series of connections would be extremely valuable to the designer. It would result in economy for both designer and fabricator. It would eliminate many of the present problems. The clip angle connection presented here is suggested as a standard beam-to-column or beam-to-girder connection.

ACKNOWLEDGMENTS

The shear friction concept owes much to several people. Its origin is unknown. The basic idea of the clip angle connection is due to R. F. Mast. The numerous ad hoc tests to prove connection designs, and much helpful criticism on past designs, are due to Concrete Technology Corporation, Tacoma, Wash.

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2. Mast, R. F., "Auxiliary Reinforcement in Concrete Connections," *Journal of the Structural Division*, ASCE, Vol. 94, No. ST6, Paper 6002, June 1968, pp. 1485-1504. This is a "how to design it" paper. It gives many quantitative examples (composite beam, corbel, end bearing, lapped splices, confinement) showing the wide application of shear friction. It also outlines the limitations of shear friction, particularly the tension problem in beam-to-column connections.
3. Birkeland, P. W. and Birkeland, H. W., "Connections in Precast Concrete Construction," *American Concrete Institute Journal*, Vol. 63, No. 3, Mar. 1966, pp. 345-368. The paper shows many of the ways to tie a precast building together so that it will behave as well as a cast-in-place structure. It also shows other types of joints, such as a knife plate bolted beam-to-column connection.
4. "Proposed Revision of ACI 318-63 Building Code Requirements for Reinforced Concrete," *American Concrete Institute Journal*, Vol. 67, No. 2, Feb. 1970, pp. 77-186. Section 11.15 (p. 115) and pp. 168-169 give the shear friction concept proposed for the new Code. Fig. 11-E10 on p. 169 and the accompanying commentary are particularly helpful.
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6. PCI Committee on Connection Details, "Summary of Basic Information on Precast Connections," *Journal of the Prestressed Concrete Institute*, Vol. 14, No. 6, December 1969, pp. 14-58.