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Connections in Precast Concrete Construction

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Outlines and discusses requirements for connections in precast concrete buildings, and shows examples from completed structures where these requirements have been met. A time-proven hypothesis explaining shear behavior at concrete to steel and concrete to concrete interfaces is presented. Examples illustrate beam to column connections using this hypothesis. Suggestions for further study are outlined.

Key words: beam; column; connection; continuity; design; diaphragm; precast concrete; shear-friction hypothesis; shear strength.

■ SINCE 1951 THE AUTHORS' FIRM has had considerable experience in precast concrete construction of varied types. Additionally, through its close association with a precasting firm, it has also had a unique opportunity to observe connection problems and their solutions from the vantage point of the constructor. This experience has led to a connection philosophy which favors hard and continuous connections. Hard connections are those in which tensile, compressive, and shear stresses are taken by hardware, rebars and concrete, so that the deformational characteristics are similar to monolithic construction. In contrast, an example of a soft connection would be one with a neoprene bearing pad. A discontinuous connection would be, for instance, a bearing type support that depends on friction due to gravity for stability.

Many precast concrete structures have shown distress as a result of inadequate attention to connections, and the excellent AISI and ACI papers[1,2] on the 1964 Alaskan earthquake make this evident. Connections are a critical part of precast construction. Unfortunately, it is all too easy to devote one's attention to design of individual members, leaving connections until after the structural concept is frozen. It is the authors' conviction that members and connections go hand in hand, and together become the structural concept.

DESIGN GROUND RULES

It is essential to consider connection problems and their ramifications at all stages of the work, from conceptual studies through construction. In order of concern, the requirements of precast concrete connections are: structural adequacy, architectural function, and economy.

Structural adequacy

The ground rules defining structural adequacy must include satisfactory behavior at both working and ultimate loads. At service loads, the structure should be free from excessive cracking and deflections. When approaching ultimate load, the structure should behave in a ductile manner. Ideally, the individual members should yield and fail before their connections.

Adequacy also implies that the structure should be capable of carrying the loads which do actually occur, but which are not subject to a completely rational analysis. Examples are: seismic motions; thermal, creep, and shrinkage forces; differential settlement; and accident. Loads of this type have given little trouble in well-designed monolithic cast-in-place structures, for which 60 or more years' experience has shown the need for a high level of continuity. The definition of continuity must include both the moment and tension varieties. The latter may be thought of as "resistance to being torn apart." The authors are concerned about the deficiency in tensile continuity in many present precast structures, which can turn an otherwise sound structure into a house of cards. The AISI Alaskan earthquake paper shows several examples. It is the authors' conviction that it would behoove us to design precast structures to the same general level of continuity provided as a matter of course in good monolithic design.

Good precast design follows a basic precept of good monolithic design: To avoid failures, all potential failure planes must be crossed by steel. In precast work, these planes may be either between elements, or inside an element. For the former, the connection itself must provide the necessary strength. For the latter, the steel is usually rebar, for which it is essential that adequate anchorage be provided on both sides of the potential failure plane under consideration. A common source of trouble is insufficient lap between connection hardware anchors and the main structural steel. Another common trouble occurs when this lap is inadequately confined, resulting in splitting or spalling of the concrete. Rebar hoops used for confinement are inexpensive, considering the good they do. A third cause for trouble is anchor bars for hardware oriented so that they do not have the opportunity to pick up tensile stresses. This has been particularly true of bearing hardware for corbels. In most instances where connection distress has been observed, the distress could have been avoided by applying the above design precepts.

At the irreducible minimum, the structural designer's responsibility is that of conceiving and carrying through the total structural scheme of how vertical and lateral loads are carried. Connections are a highly critical part of any load path and the designer should therefore exercise

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complete control over connection design. The surest way for him to accomplish this is to detail all connections thoroughly. Experience has shown that delegation of this design responsibility to others may lead to argument, sleepless nights, and possible disaster.

Architectural function

It is in the nature of things that the architect must be sure of his own design concept before he can sell his client on it. After this is done, any change in concept comes hard, if at all. Since connection design directly affects both over-all and detail architectural function, it is important that the structural designer be consulted early in the conceptual stage.

Precast buildings are often built because they are more economical to construct; certainly not because they are easier to design. They are also built when there is an architectural requirement for precise and complex formed surfaces. In general, precast buildings can be made more economical only when advantage is taken of the smooth and precise surfaces obtainable in factory production. These may be exposed, and need not be disguised or hidden behind plaster, thus leading the architect to full expression of the structure.

Considering the detail requirements of architectural function, it is desirable that connections be neat and clean, essentially invisible, and compact. It is important to avoid unwanted shadow lines, cracks, awkward shapes, intrusion into living space, and the like. The corbel is an example of conflict with architectural function.

Economy

Over-all project economy requires both ease of erection and ease of fabrication. Many fundamental erection problems can be recognized and solved if the designer erects the building completely on paper, during early preliminary work and before the structural concept is frozen. This practice, if supplemented with talks with local erectors and fabricators, will avoid many common problems with where to put large erection equipment, placing sequence of precast elements, absorption of fabrication and erection tolerances, and rebar or hardware interferences. Fabrication problems are normally easier to recognize, with local fabricators usually able to offer constructive criticism.

However, it is human nature for the erector to want fancy hardware which will make his job easier, but which costs much more than the money saved in erection. Similar comments can be made concerning the fabricator, the general contractor and others. The designer must arrive at the solution involving the least over-all project cost. It is important for the designer to realize that good precast design must reflect the constructor's problems to a considerably greater degree than is required with other materials.

The following comments concerning economy apply both to connections and to over-all design: Make precast elements as large and as few in number as possible, but as limited by available fabrication, transport, and erection facilities. This saves in form turnover, handling, erection, and makeup of field connections. Make the buildings all precast. Or, if this is not feasible, make the precast sections separate from the cast-in-place. This drastically saves erection time and the owner's interest money. Make connections such that the crane can place a piece and be freed immediately, with completion of the full structural connection waiting for a more convenient time. Make temporary connections with bolts, in preference to welds. Cover connection hardware and steel with drypack (no forms needed) or with cast-in-place concrete. Make designs using the latter such that they are either self-forming, or use bolt-on forms. Make beams and slabs composite, but design so that shoring is not needed.

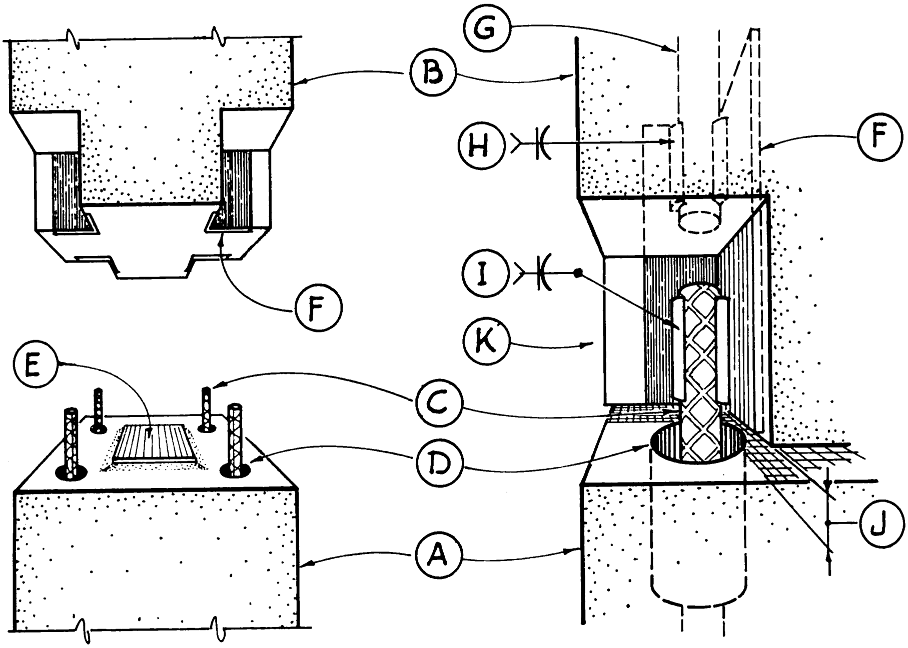
CONNECTION EXAMPLES

Certain connections have been developed which are felt to satisfy the ground rules outlined above. Examples of these connections are: column to footing, column splice, beam to column, floor slab to beam, floor diaphragm, and tensile continuity. Connections similar to those described herein were used in the Washington State University dormitories pictured in Fig. 1. These buildings were completed in 1964. They are entirely precast, from top of column footings through to roof, except for the end stairs and central core.

Column to footing connection and column splice

A standard connection joining a precast column to cast-in-place footing is shown in Fig. 2. The dowels are set by template, with sleeves to allow some lateral adjustment. The bearing plate is set level and to accurate elevation on a grout pad. The column is lowered onto the plate and guyed or braced, thus freeing the crane quickly for other operations. The dowels are then welded to the embedded angles. Drypack is rammed into the gap under the column and into the corner blockouts, thus completing the connection. This detail provides full structural continuity, gives the flush appearance of monolithic concrete, is fully

Fig. 1 — Precast dormitories (Washington State University)



A. Cast-in-place footing
B. Precast column
C. Dowels
D. Sleeves
E. Bearing plate
F. Embedded angle
G. Column rebar
H. Shop weld
I. Field weld
J. Gap
K. Corner blockout

Fig. 2 — Column to footing connection

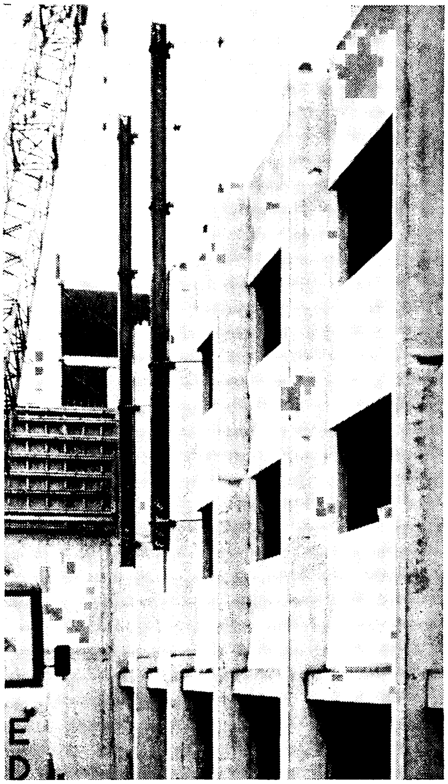


Fig. 3 — Column erection (Washington State University dormitories)

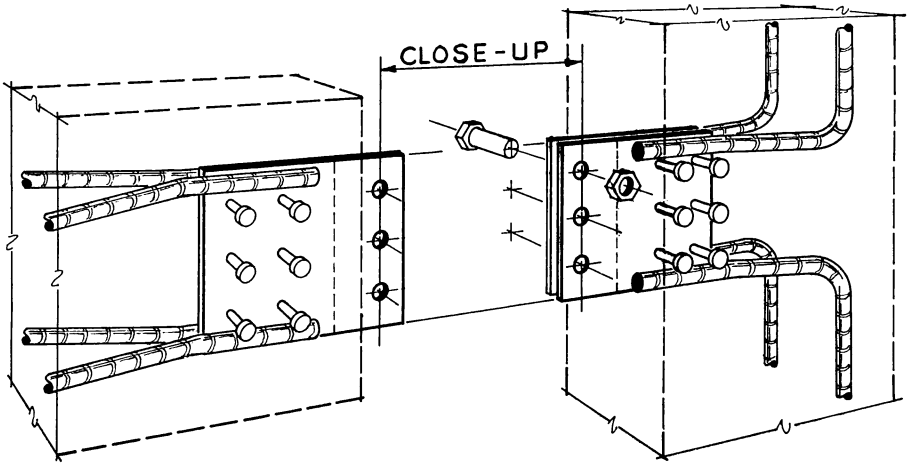


Fig. 4 — Knife connection

fireproof, is easy to erect and fabricate, and will absorb any reasonable lateral or vertical tolerances in the cast-in-place work. Strength analysis is conventional.

Where handling and erection problems limit column lengths, a splice similar to the above is suitable. The dormitories shown in Fig. 1 required splices in the exterior columns in the fourth and seventh stories. Fig. 3 shows a portion of one building ready for erection of the story four to seven columns, with two temporary strongbacks (vertical steel beams) in position.

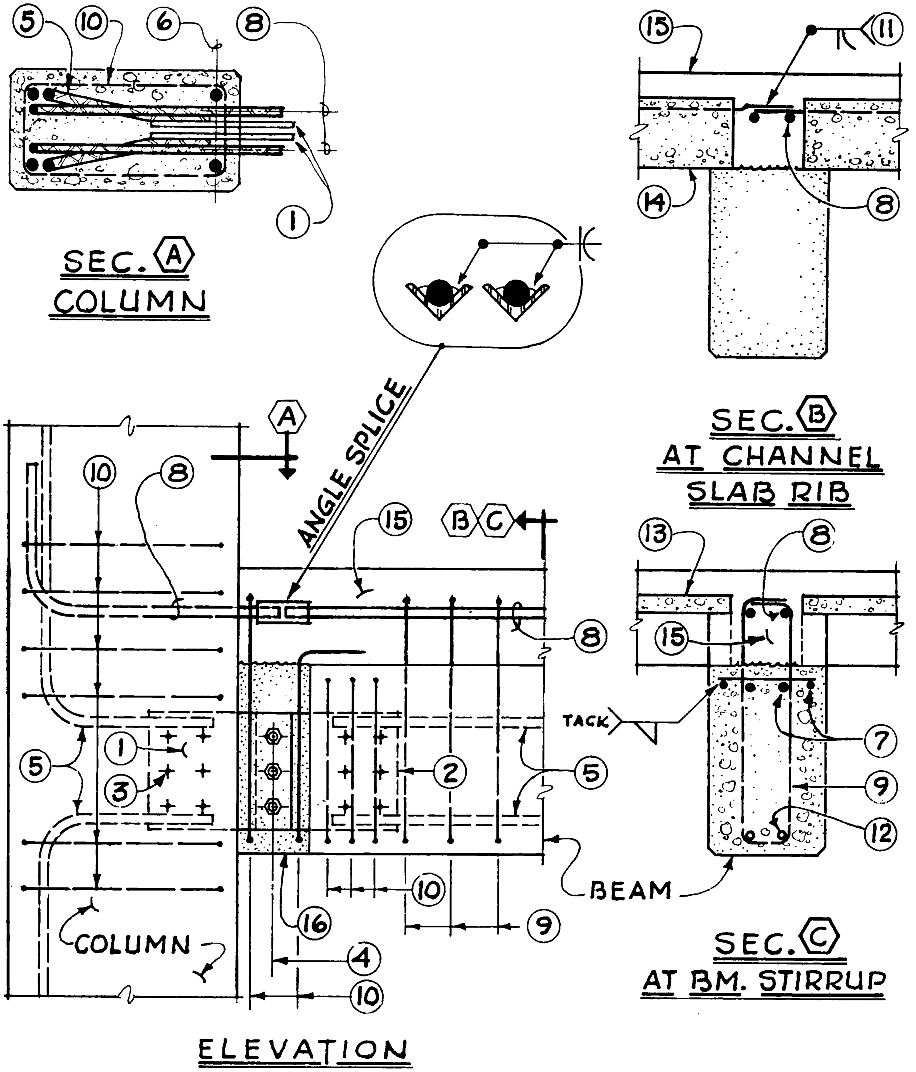
Beam to column connections

Structural continuity and ease of erection comprise the two chief conflicting requirements of beam to column connections. Attaining reasonable erection cost means that the connection must permit the crane to be freed immediately after the beam is placed.

A common floor beam detail used by the authors' firm is the knife connection. This connection was developed to avoid the poor appearance of corbels and the lateral clearance (needed during erection of beams) requirements of notches. Fig. 4 is a perspective sketch showing arrangement of hardware. Fig. 5 is a schematic showing the basic system of reinforcement, and how the precast pieces are held together. The two stubs projecting from the column face are the ends of the anchorage rebars for the beam's negative moment steel. The space between these stubs allows the beam to be lowered into place, with no interference between beam blade and the stubs. The beam blade slides between the two plates which form the column sheath. Alternatively, the beam may be raised into place. Erection bolts are inserted, freeing the crane. The holes at one end of the beam are normally punched undersize. These are now field reamed to absorb fabrication and erection tolerances, and structural bolts are installed. The rebar for negative moment continuity is now set, the floor slabs placed, and the continuity welds made for both beam and floor slab negative moment. The ends are drypacked and the fill concrete cast, completing the connection.

Considering structural requirements, the knife connection provides full continuity for superposed dead and live load negative moment. This continuity steel, together with the knife hardware, makes an excellent tension tie in the transverse direction (parallel to the beam). Several ad hoc tests for widely varying applications have shown very good behavior at working load, and excellent, ductile behavior at ultimate load. Loadings have included large torsional and tensile forces. Shear strength analysis is not conventional, and is based on the shear friction hypothesis discussed in a later section. A sample connection analysis is also given.

Considering other requirements, the knife connection provides the neat and clean appearance of monolithic concrete, is fireproof, economi-



1. Sheath plates
2. Knife plate
3. Headed studs
4. Bolts
5. Anchor bars
6. Column rebar
7. Crack control and edge rebar
8. Negative moment rebar
9. Stirrups
10. Confinement hoops
11. Channel slab continuity splice
12. Prestress strand
13. Channel slab
14. Channel slab rib
15. Topping and cast-in-place concrete
16. Drypack

Fig. 5 — Knife connection details

cal to fabricate and erect, and will absorb the usual tolerances in column spacing and beam length. Service experience, covering many applications, has been uniformly excellent.

Floor slab to beam connections and floor slab continuity

Floor slab to beam connections should satisfy two structural requirements: beam action composite with the floor slab, and slabs continuous in tension across the beam.

A standard detail used for channel slab floors is that shown in Fig. 6. The channel slabs bear on the top edges of the beam. The space between is to be filled with cast-in-place concrete. Into this space projects top steel from each channel slab rib. Each projecting rebar stub is field welded to the matching rebar from the channel slab opposite. Projecting upward into this space are the beam stirrups, which enclose the negative moment steel for the beam. The sections in Fig. 5 show both channel slab rebar and the beam steel. Following placement and welding of the beam negative moment steel, and welding of the channel slab rebar, the space is cast full of concrete.

For floor construction, a concrete topping is then placed. This topping is useful for hiding electrical conduit, etc. Consequently, its structural value cannot always be depended on, and it is often considered as nonstructural dead load. Where channel slabs are prestressed, slippage inside the forms upon release make end diaphragms uneconomical to produce by precasting. However, the forming system shown in Fig. 7 makes it possible to cast the end diaphragms in place, after erection, using the beam fill concrete. It produces the end diaphragm with a smooth surface flush with the beam face.

This connection detail is architecturally pleasing, since it permits full exposure of the ceiling. It satisfies the structural requirements of beam action composite with the floor, and of slab moment and tensile con-

A. Beam
B. Opposite channel slabs
C. Projecting top rebar
D. Field weld for tensile and moment continuity

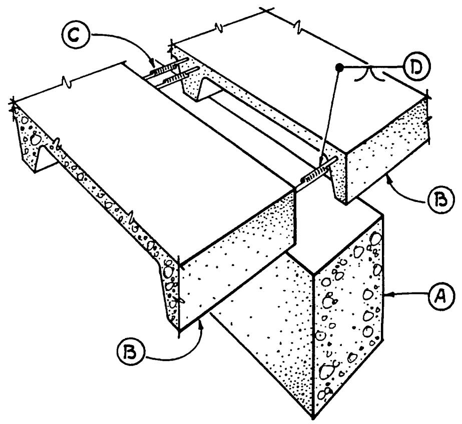
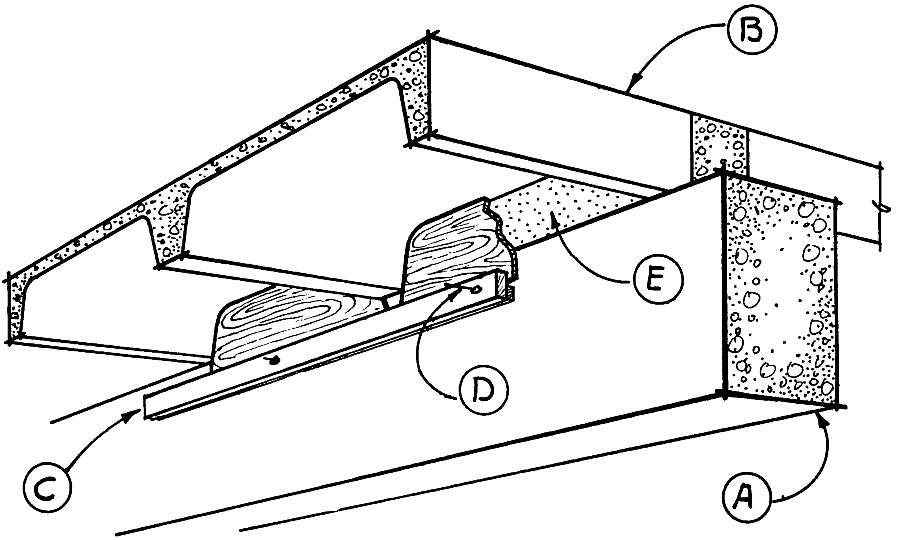


Fig. 6 — Channel slab continuity



A. Beam
B. Channel slab
C. Wood closure form bolted to beam
D. Screw inserts for forms
E. Cast-in-place fill concrete flush with beam face, after removal of closure form

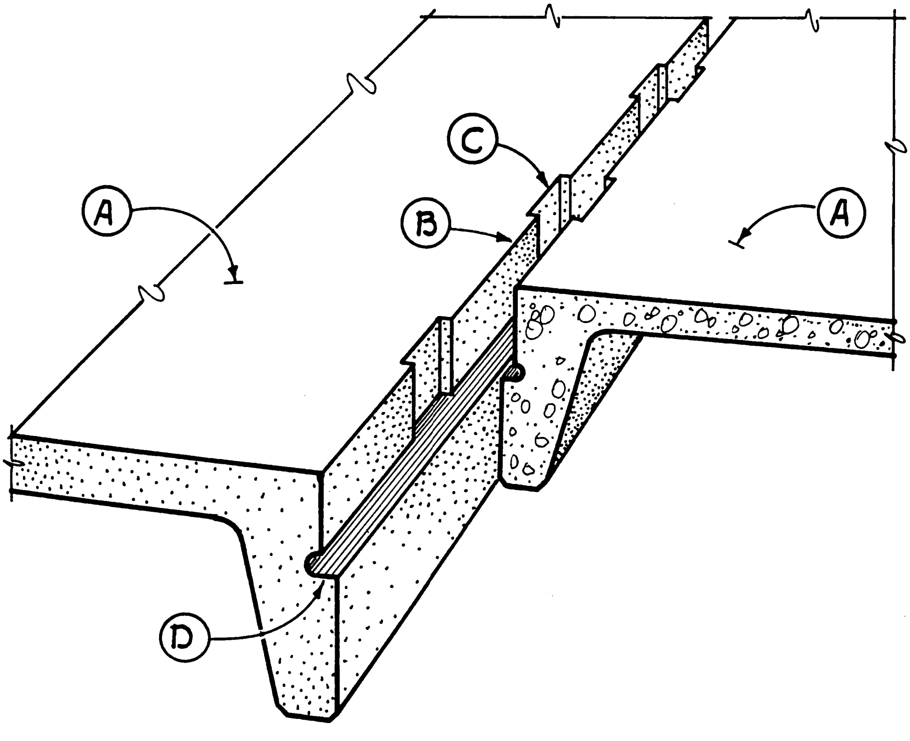
Fig. 7 — Channel slab end closure

tinuity. The latter does not come free, because of the required field welding. However, other methods known to the authors, such as hooking the projecting rebar from opposite channel slabs in the fill concrete, are much softer connections, and have shown distress. Where this hooked rebar provides the only longitudinal tie holding the building together, the soft connection can easily crack or come apart under thermal, creep, shrinkage, or racking deflections. The authors feel that this is one place where a little extra cost is justified in better performance.

Diaphragm connections

Connections to obtain floor and roof diaphragm action should be designed with special care, since they are essentially nonductile. Joints between adjacent panels in a bay are of most concern. Such a joint must take shear in both horizontal (diaphragm action) and vertical (distribution of concentrated vertical load) directions. It should also maintain its integrity while preventing rotation of the member about its own axis. This may occur in certain buildings during seismic motion. A common example of poor connection is the typical single-tee roof, where shear connection between adjacent tees is provided by field welding lightly anchored embedded clips. Where embedded in the thin edges of single or double tees, these clips have shown outstanding lack of strength. The authors much prefer precast panels with thick edges, such as channel slabs, or tees with thickened top flanges. A thick edge makes reliable anchorage of embedded hardware possible. Whether by rebar or stud, this anchorage should lap main reinforcement sufficiently to obtain a tension splice.

Diaphragm shear connections can also be obtained by using shear keys (see Fig. 8). This is a standard detail used with channel slab and beam construction. The figure shows recesses along the channel slab edges which are parallel to slab span. Concrete is placed into the slot between slabs, filling the matching recesses, thus forming shear keys. Note that if integrity is to be preserved, the slabs must be restrained from moving apart, in the direction perpendicular to the joint. This restraint is provided by the beams.



A. Adjacent channel slabs
B. Slot for cast-in-place concrete
C. Matching recesses forming diaphragm shear keys
D. Key for vertical shear

Fig. 8 — Shear connection for channel slab diaphragm

Tying the building together

Cast-in-place concrete may be utilized to absorb fabrication and erection tolerances, and to provide tensile continuity to tie the individual pieces of the building together. In the transverse direction, parallel to the beams, the beams can provide this tie. The longitudinal direction is of most concern, because adequate tensile continuity does not come naturally, and must be consciously designed into the structure.

The best solution, if spandrels can be made structural, is to post-tension through the spandrels for the entire length of the building, using bonded tendons. Spandrels of this type can be seen in Fig. 1 and 3. Unbonded tendons are not recommended, since a local failure destroys the prestress over the entire tendon length. Such prestress also tends to close or prevent cracks due to thermal and shrinkage effects, thus greatly reducing the need for expansion joints in long buildings. Where spandrels cannot be made structural, arrangements similar to that shown in Fig. 9 are suitable, though less desirable. The example shown is taken from a three-story dormitory. In Fig. 9 the wide cast-in-place strips along the corridor columns provide a place for continuous rebars to run the length of the building. Anchored at the ends, these rebars comprise the desired longitudinal tensile continuity. In this example,

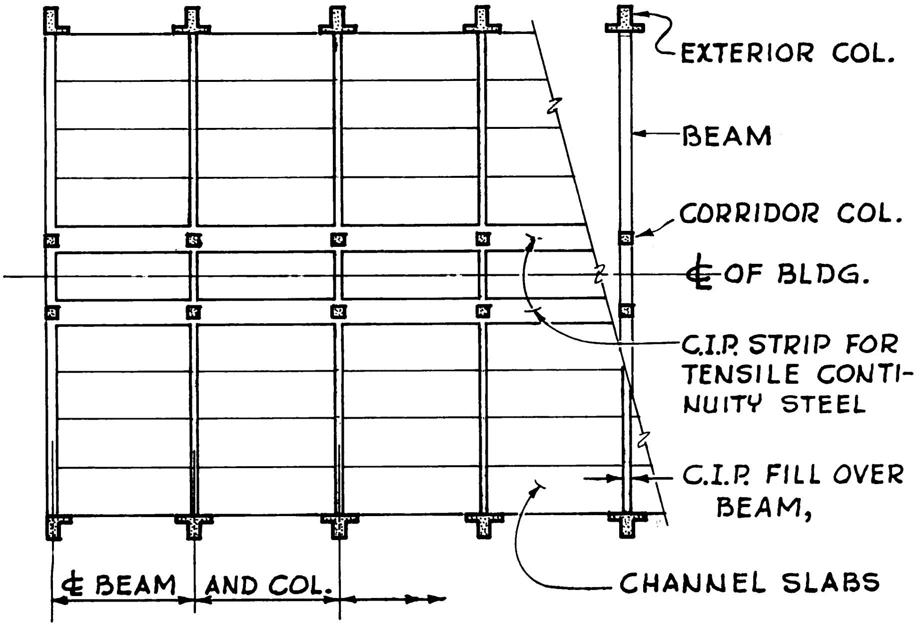


Fig. 9 — Floor tensile continuity (partial plan view)

transverse shear walls are closely spaced. Where shear walls are widely spaced, similar cast-in-place strips should be provided at the outside edges, so that the floor diaphragm can be developed as a deep beam.

Fabrication tolerances affect beam lengths, panel size and squareness, cross-sectional dimensions, positions of hardware, etc. Erection tolerances result from placement of elements, alignment errors, and so forth. Ideally, a design should be inherently insensitive to these tolerances. The example shown in Fig. 9 is insensitive. In the transverse direction, beam length variation is absorbed in reaming the bolt holes in the knife plates. Variation in the cumulative channel slab width due to bow, form growth, and placement is absorbed in the cast-in-place column strip. In the longitudinal direction, variations in channel slab length and errors in placement and erection are absorbed in the cast-in-place beam fill.

SHEAR FRICTION HYPOTHESIS

Introduction

A large proportion of the connective distress found in precast buildings is centered around the shear interfaces associated with corbels, bearing shoes, ledger beam bearings and the like. For these concrete to steel and concrete to concrete interfaces, as well as potential cracks in monolithic concrete, ordinary beam shear-flexure and principal tension analyses do not apply. Here, we are concerned with shear failure as slippage (not as a tension crack in the usual sense) along a plane of maximum shear. Examples might be a vertical plane at the upper reentrant corner of a corbel, or the interface between a precast beam and a cast-in-place floor slab. The design of heavily loaded beam-to-column connections has led the authors' office to the use of a design tool called "shear friction." After some years of use, this tool has proven extremely useful, with wide application. One of its most appealing qualities is that it permits easy visualization of structural action in a heavily loaded, compact connection.

Hypothesis

Consider the monolithic concrete block shown in Fig. 10. Assume a crack (failure plane) along m-m. The external shear loads V tend to produce slippage along the plane. The shear loads should be understood as colinear. The slippage is resisted by the friction μP resulting from the external clamping force P.

If the crack m-m is rough, sliding motion along it will cause a separation δ of the two halves (see Fig. 10). If reinforcement is placed across the interface, the separation will develop tension T in the reinforcement. Since the reinforcement is well anchored on both sides of the crack, the tension provides an external clamping force on the concrete,

TABLE 1 — HEADED STUD SHEAR COMPARISONS

Stud diameter, in.	A_s, sq in.	Allowable working shear (kips per stud)		
		Shear friction from Eq. (3)	AISC Building Code ($f_c' = 3500$)	Proprietary (Reference 6)
½	0.20	5.4	5.5	5.4
⅝	0.31	8.4	8.6	8.4
¾	0.44	11.9	12.5	11.9
⅞	0.60	16.2	16.8	16.2

resulting in compression across the interface of equal magnitude. The roughness may be visualized as a frictionless series of fine sawtooth ramps, having a slope of tan ϕ. Comparing Fig. 10(b) with 10(a), $T \tan \phi$ is seen to be equivalent to the friction force μP. The tension T is seen to produce the equivalent of the external clamping force P.

The above description is for an interface (crack) in monolithic concrete. The concept can be extended to artificially roughened or smooth construction joints, and to concrete to steel interfaces. For these, smaller values of tan ϕ are appropriate. Note that this concept develops shear by friction, not by bond. The reinforcement across the interface is stressed in tension, and with dowel action presumed insignificant.

Referring to Fig. 10(b), the ultimate shear capacity V_u across the interface will be reached at yield of the reinforcing ($T_u = A_s f_y$). The ultimate shear can be written as follows, with suggested tentative design limits and tan ϕ values shown below:

$$V_u = T_u \tan \phi = A_s f_y \tan \phi \quad (1)$$

$$v_u = V_u / A_g = p f_y \tan \phi \quad (2)$$

where

tan ϕ = 1.7 for monolithic concrete

tan ϕ = 1.4 for artificially roughened construction joints

tan ϕ = 0.8 to 1.0 for ordinary construction joints and for concrete to steel interfaces

V_u = total ultimate shear force

A_s = total crossectional area of reinforcing across interface

f_y = yield strength of reinforcing (≤ 60 ksi)

A_g = gross area of interface

v_u = ultimate shear stress on gross area (≤ 800 psi)

p = steel ratio, A_s/A_g (useful limit $= 0.015$)

Concrete strength, $f_c' \geq 4000$ psi

Maximum reinforcing size = #6 rebar, or ½ in. diameter headed studs

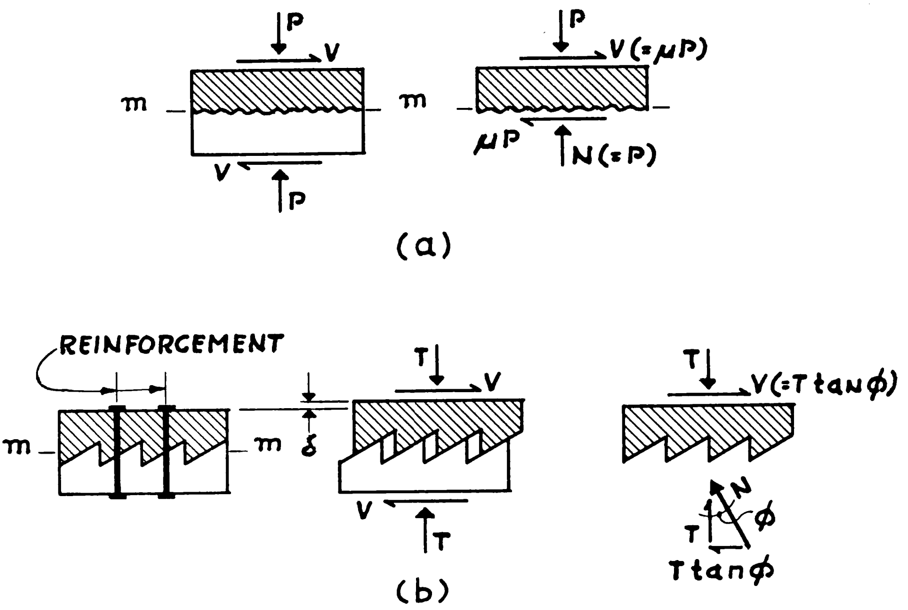


Fig. 10 — Shear friction hypothesis

Several cautions are in order. Reinforcement must be anchored on both sides of the failure plane, sufficiently well to develop yield in the steel. The concrete must be well confined, with liberal use of hoops required. Headed studs should engage as much concrete as practicable and necessary. Lengths should be long enough to avoid potential failure planes beyond the stud heads. Tests by Laing[7] of short heavy studs indicate this to be a real consideration. External tension loads decrease the clamping force, and must be accounted for. The interface must be sound and free from laitance, sawdust, paint or loose rust. Care must be exercised in sketching all possible failure planes, and in providing sufficient well-anchored steel across these planes.

Discussion

A considerable body of data may be interpreted as supporting the shear friction hypothesis. Shear strength values for pushoff tests and for horizontal shear failures in large scale girder tests are plotted in Fig. 11. Pushoff specimens are shown in Fig. 12. The rebar used in all these specimens was ASTM A-15, Intermediate Grade, with actual yield strengths running about 50 ksi. Hanson's pushoff tests[3] were primarily intended to evaluate the effect of different surface finishes on the horizontal shear strength at the interface between a precast girder and a cast-in-place deck. Consequently, effects of steel percentage were not investigated systematically. Data (Table 1, Hanson) were given with

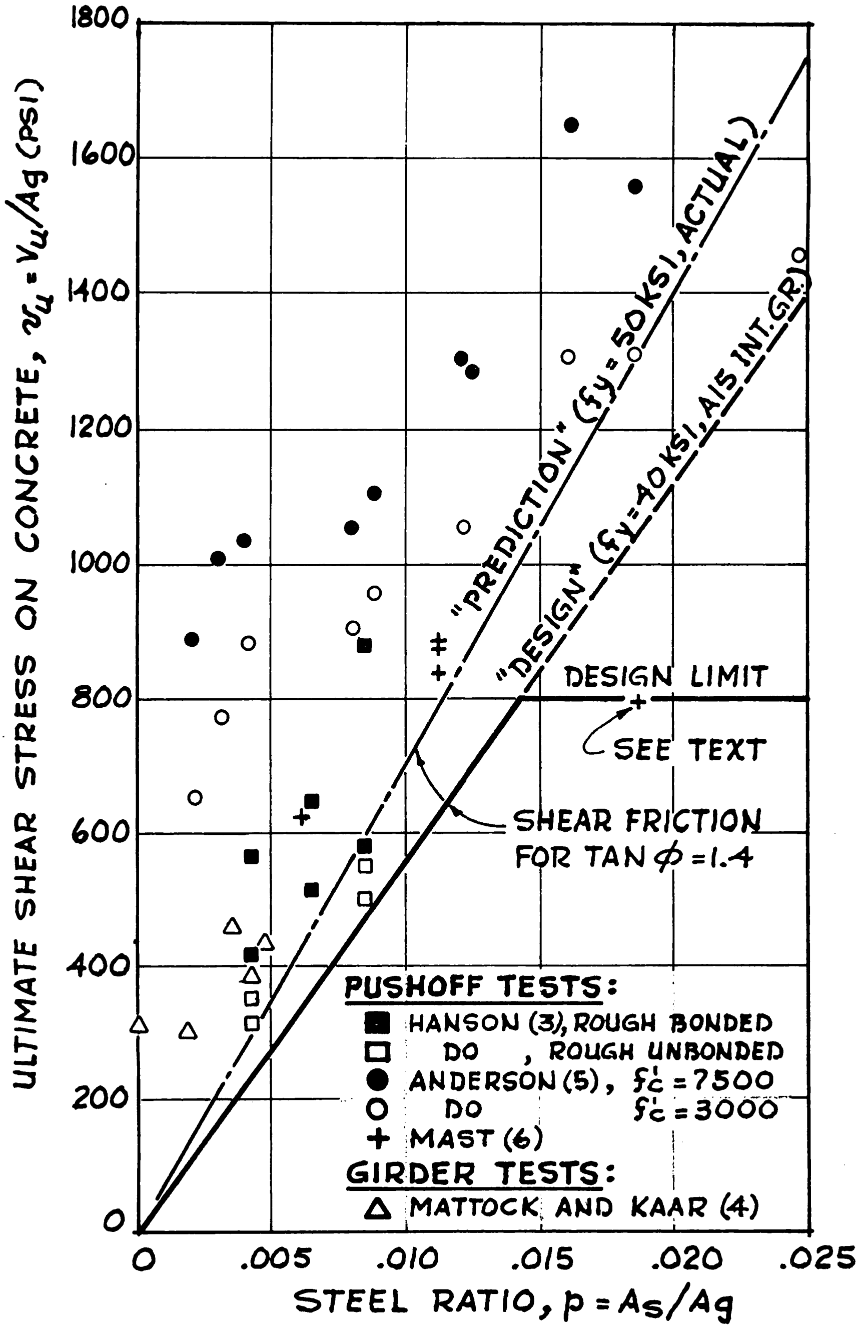


Fig. 11 — Horizontal shear data

stirrup effects subtracted. By adding the stirrup effect (Fig. 4, Hanson) back in, it is possible to compare the data with shear friction predictions. This was done for representative rough-bonded and rough unbonded specimens. Fig. 11 shows that only two unbonded specimens fell below the prediction line, which is based on the average yield strength for the rebar used. These two points fall slightly above the design line, which is based on the minimum specification yield strength. Agreement appears good.

Mattock and Kaar (Reference 4, Table 6) summarize horizontal shear stresses attained in several test series of precast girders with cast-in-place composite decks. For those girders which failed in horizontal shear, the maximum interface shear stresses are plotted in Fig. 11. These points are at low steel percentages, making bond dominant over stirrups.

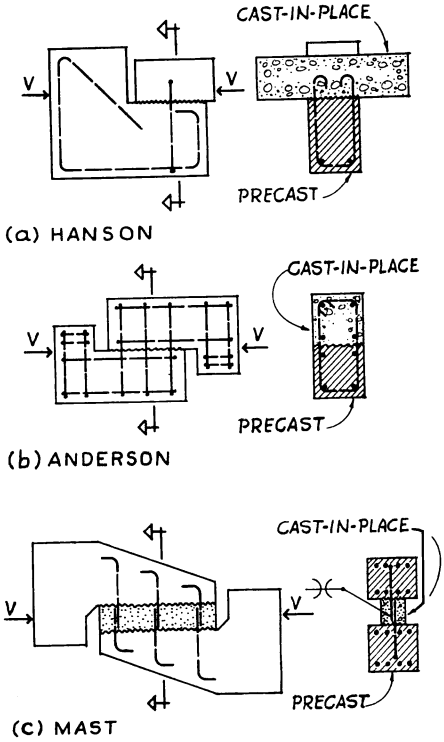


Fig. 12 — Pushoff specimens

Anderson (Reference 5, Fig. 3 and 4) gives pushoff data for specimens intended to simulate building connections. Rebar was well distributed and very well anchored. The data are replotted on Fig. 11. Note that the 3000-psi concrete attained 1450 psi shear ($26\sqrt{f_c'}$) and that the 7500-psi concrete reached 1600 psi ($18\sqrt{f_c'}$). Maximum rebar size was #5.

Unpublished work by Mast* was intended to prove the design of horizontal shear connections between the precast longitudinal strips of a barrel shell roof.† These strips had formed edges, with roughness provided by saw-tooth form bulkheads. Rebar projecting into the gap between strips was lapped and welded, and the gap filled with cast-in-place concrete. The specimen shown in Fig. 12(c) simulated this connection. Strength data are plotted in Fig. 11. Unfortunately, the one specimen with a high steel percentage was tested when the cast-in-place strip was only 1½ days. The concrete strength for this particular specimen is unknown, but was undoubtedly very low. It is felt that the strength otherwise would have fallen much nearer the shear friction

*Mast, R. F., "Shear Tests of Barrel Shell Joints," Nov. 1962 (Unpublished data).
†Pier 46 Transit Shed, Port of Seattle, Seattle, Wash.

line. Because of this one specimen, we have limited our designs to $v_u = 800$ psi and a useful limit for p of 0.015. The other specimens were tested at ages producing cylinder strengths of 3000-5000 psi.

A large amount of testing has been done by the steel people on horizontal shear connections (headed studs) for steel beams composite with cast-in-place floor slabs. Their conclusions, based on ultimate strength concepts, are stated as allowable working loads in Table 1.11.4 of the 1963 AISC Building Code. Proprietary data[6] have also been published. Strengths agree closely with shear friction calculations, which are as follows:

Assume $f_y = 0.9\ f_u = 54$ ksi; factor of safety (FS) $= 2.0$, and $\tan\phi = 1.0$ (smooth interface). Then

$$V = \frac{V_u}{\text{FS}} = \frac{A_s f_y \tan\phi}{\text{FS}} = 27\,A_s \qquad (3)$$

Numerical results, with comparisons, are listed in Table 1.

Arbitrarily limits were chosen on gross shear v_u, steel ratio p, concrete strength f_c', and reinforcing yield f_y, for use in our own work. These limits are given following Eq. (2), and cover regions where our test and service experience has been good. Numerous ad hoc tests of production connections have all shown that the shear friction hypothesis predicts a lower bound on connection strength. Behavior of these connections in tests has been excellent, both at working and ultimate loads. To simulate accidental loss of bond (a potential problem in buildings, due to rough erection practice and other possibilities), several tests have been run with deliberately destroyed bond. No significant differences in behavior have been noted.

Concluding, it is felt that the shear friction hypothesis is an extremely useful tool for connection design. It is readily understood and remembered. It is simple and easy to apply. Its use would have prevented most cases of connection distress that we have observed. However, the data presented do not directly verify the hypothesis. A test program specifically planned to verify the hypothesis (or torpedo it) would be of great interest. It is possible that other, simple hypotheses could explain the supporting data as well as does shear friction. If so, this would provide a good reason for an enlarged test program. To be willingly adopted by the practicing designer, any such hypothesis must be simple. Simplicity is not easy to attain. A case in point is the beautifully simple equation [second part of Eq. (16-1), ACI 318-63] which states the ultimate moment capacity of a beam as:

$$M_u = \phi\, A_s f_y (d - a/2)$$

This equation can be quickly derived from simple statics, is rational, and can be easily remembered.

EXAMPLES

Design examples of two types of beam to column connections are shown. The first is a knife connection, the second a bearing connection. They show the application of the shear friction hypothesis with the essential calculations. Both connections have been used successfully in structures, and similar full scale connections have been tested on an ad hoc basis to ultimate load.

Knife connection

A review of a two-sided knife connection between prestressed beams and a column is made. Working stress design is used. Each side carries a total service reaction of $V_{D+L} = 77.5$ kips. Dead and live load parts are equal. Details of the connection are shown on Fig. 13.

Beam part of connection—It is assumed that the moment at the bolt line is zero. The vertical shear is taken by studs (shear friction), and the moment by anchor bars. The load on one stud is 4.9 kips, which is to be compared with the allowed load of:

$$A_s f_s \tan \phi = 0.20 \times 27 \times 1.0 = 5.4 \text{ kips}$$

Other calculations are conventional, and include bolts in shear and bearing, bending of plate, and direct stresses, bond anchorage and welds of anchor bars. It may be necessary to review the plate for torsion if the beam is loaded unsymmetrically during construction, and if torsional restraint is not provided at that stage.

Column part of connection—The most severe condition is obtained with dead load on both sides of the column, but with live load only on one side. The stud group is handled the same as a rivet group under eccentric load, i.e., for each stud the loads due to vertical shear and moment are added vectorially. The maximum resultant on any stud is then 5.1 kips, compared with the allowable 5.4 kips.

Comments—The stud steel ratio is somewhat higher than the suggested limit ($p = 0.015$) since ad hoc tests of similar designs showed excellent behavior, the higher steel ratio is considered acceptable. The following comments supplement Fig. 13, and are keyed thereto:

1. Action of studs against concrete is upward. Therefore, hardware is placed as low as practicable in the beam. Location should also be coordinated with prestressing strands.
2. Beam rebars are not shown. There must be hoops and stirrups that fully enclose hardware and strands.
3. Column rebars are not shown. There must be ties snug to plates top and bottom.
4. Spacers shown are minimum to hold column plates in relative position. Interior spacers may be added, though they are not required by shear friction hypothesis.

5. Dowels projecting from column, for connection to negative beam rebars, are not shown.
6. Bend in anchor bar is placed some distance in from end to avoid possible embrittlement resulting from the quenching effects of welding on the cold bent region. Anchorage length is made generous. Bars are flared as much as possible at ends.
7. After connection is bolted, hoops must be placed in the space between beam and column, and the space filled with nonshrinking drypack or concrete. This has not been shown.

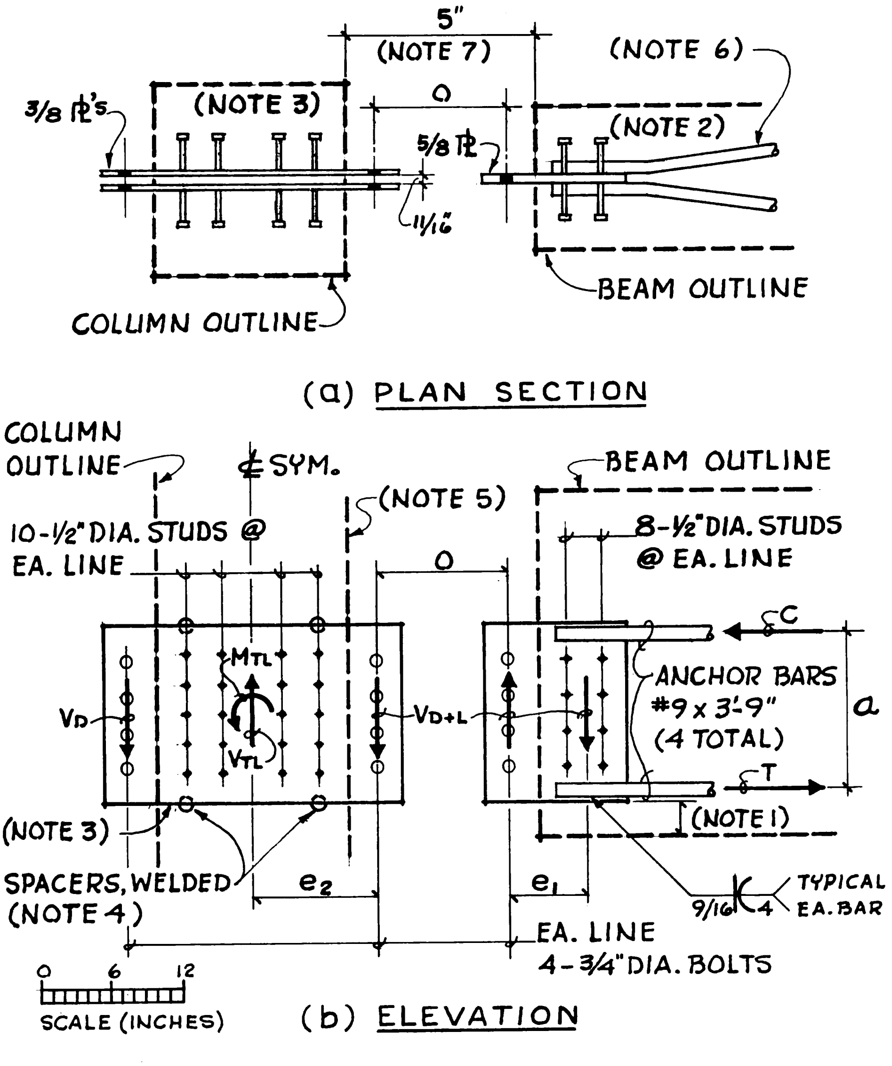
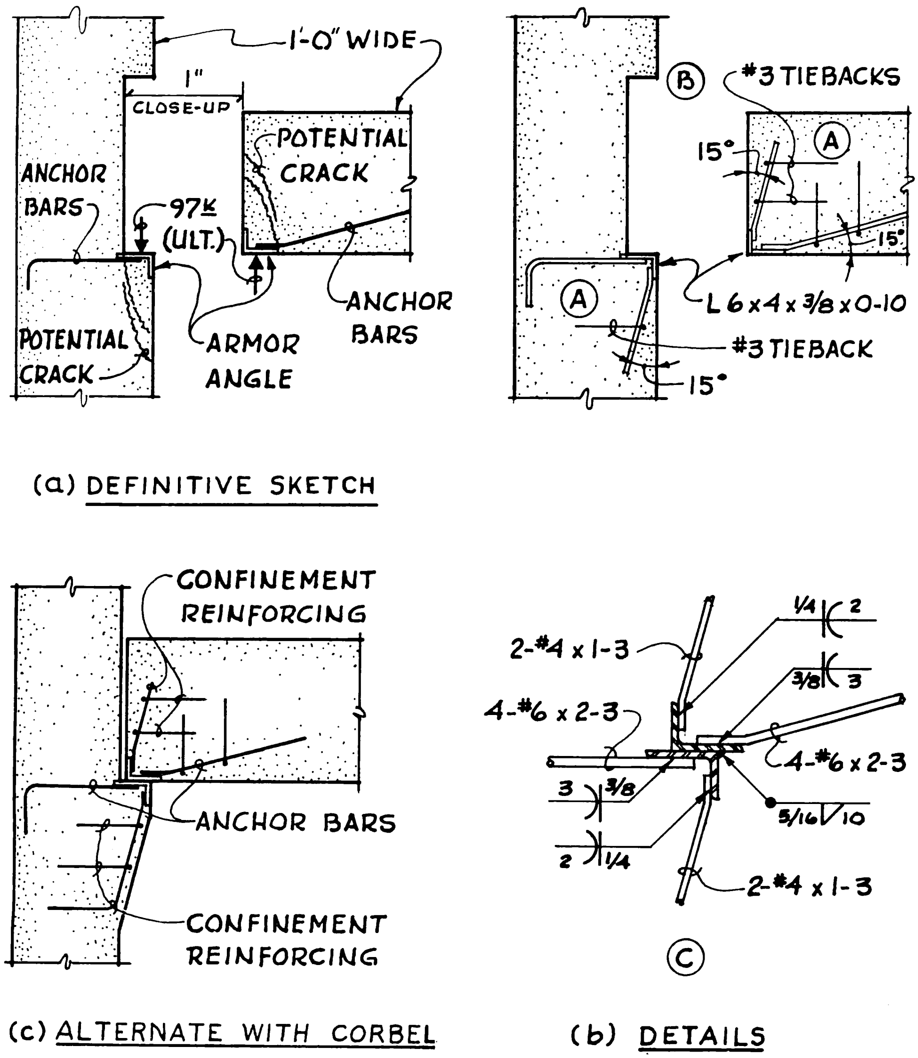


Fig. 13 — Knife connection example

Bearing connection

A design of a bearing connection between a prestressed beam and a column is made. Ultimate strength design is used. The ultimate load reaction is 97 kips. Definitive sketch and details are shown on Fig. 14(a) and (b), and an alternate with corbel on (c).



A. Additional confinement provided by column ties and beam stirrups
B. Negative reinforcement is part of total connection
C. Enlarged detail of welds

Fig. 14 — Bearing connection example

Beam part of connection—The load is taken by the armor angle and anchor bars (shear friction). The potential failure crack indicated on the definitive sketch is crossed by the bars, which are anchored at their outer ends by the angle. The required area of the bars is:

$$A_s = \frac{V_u}{\phi_v \, f_y \tan \phi} = \frac{97}{0.85 \times 40 \times 1.7} = 1.68 \text{ sq in.}$$

The required concrete area at the potential crack is obtained from the limits presently proposed for steel ratio and for ultimate shear stress, which indicates that the crack should not be shorter than 12 in. From inspection of the definitive sketch, it appears that the crack will not be shorter than this.

Column part of connection—Design is the same as for the beam part, except that it is rotated 180 deg.

FUTURE WORK

The shear friction hypothesis should be thoroughly verified and evaluated with a laboratory quality test program. If the hypothesis is disproven, an alternate, simple physical model of behavior should be developed to explain the data. This physical model should be considered acceptable if it will consistently predict a lower bound on connection strength. High accuracy is not required. The model should be applied to the design of typical connections such as corbels, corner armor, knife connections, etc., and tested.

It would also be desirable to include in the program specimens that have common construction deficiencies built in. Minimum limits should be determined and explained for the thickness of steel plate in relation to stud diameter, and for the spacing of studs. It appears that length of stud, when applied to steel plate, need not be based on the usual bond-anchorage (pull-out) approach, and an explanation for this may be sought.

Better framing concepts for high rise precast buildings should be developed, to eliminate the current dependence on shear walls or cast-in-place cores for primary resistance to lateral loads. The objective should be to obtain behavior comparable to moment-resisting frames in steel and cast-in-place concrete, and thereby allowing very tall precast buildings in seismic regions.

Improved connection details are needed. This might include making the knife connection suitable for moment reversal in seismic sidesway, and in developing a reliable shear connection between single or double tees. There also appears merit in testing the application of shear friction to composite beams whose stirrup spacing is limited by interface shear.

ACKNOWLEDGMENT

The philosophy of connection design presented here is the joint development of many people. The partners and employees of the authors' firm have had a prime role. The thought of using headed studs with knife connections came from practicing engineers who attended a course in prestressed concrete design at the University of Washington. Some experimental work was completed recently by Laing.[7] The germ of the idea for shear friction is believed to have originated with Dr. Hubert Rüsch, and was brought to light in the firm's office by Dr. Ernst Basler in 1958. Embryonic presentations of the shear friction hypothesis have been made in References 8 and 9. The numerous ad hoc tests to prove connection designs, and the courage to use novel solutions in alternate design competitions have come through the associated precaster, Concrete Technology Corporation. Some of the connections shown here were developed for buildings designed by the Tacoma architectural firm of Lea, Pearson and Richards, AIA. This firm was responsible for the dormitories shown in Fig. 1.

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Sinopsis—Résumé—Zusammenfassung

Conexiones en Elementos de Concreto Precolado

Se esbozan y discuten los requisitos para conexiones en edificios de concreto precolado, y se muestran ejemplos de estructuras completas donde estos requisitos han sido cumplidos. Se presenta una hipótesis probada por el tiempo que explica el comportamiento a cortante de las superficies de contacto entre concreto y acero, y entre concreto y concreto. Se incluyen ejemplos que ilustran conexiones de vigas a columnas usando esta hipótesis. Se presentan sugestiones para estudios futuros.

L'Assemblage des Eléments en Béton Prémoulé Utilisés en Construction

Cet article traite des exigences auxquelles doivent satisfaire les techniques d'assemblage des éléments en béton prémoulé utilisés dans la construction des édifices: on y cite plusieurs exemples de structures actuellement érigées où ces exigences ont été respectées. On soumet une hypothèse, d'ailleurs confirmée par le temps, à partir de laquelle on explique le mécanisme du cisaillement au droit des surfaces de contact béton-sur-acier et béton-sur-béton. On décrit quelques types d'assemblages poutre-à-colonne concus en s'appuyant sur cette hypothèse. On souligne également certains aspects du problème qui pourraient faire l'objet d'études subséquentes.

Verbindungen im Fertigbetonbau

Umreisst und diskutiert die Erfordernisse an Verbindung in Fertigbetonbauten, und zeigt Beispiele von vollendeten Bauten, wo die Erfordernisse erfüllt waren. Eine zeitbewiesene Hypothese, die das Schubverhalten von Beton-zu-Stahl und Beton-zu-Beton in den Verbundflächen erklärt, wird dargebracht. Beispiele illustrieren Balken-zu-Stützen-Verbindungen unter Benutzung dieser Hypothese. Vorschläge zu weiteren Untersuchungen sind umrissen.