Connections Between Precast Beam to Column Members

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One of the most useful characteristics of a precast concrete structure is that it may be designed to act in virtually any manner the designer wishes.

A precast structure is composed of individual pieces, but they may be connected by any number of different methods. These connections determine the type of action developed in the members and in the structure as a whole. Actually, it is the connections that determine whether the structure will act as a unit, or as a series of separate members.

For example, a precast frame allows the designer to choose between continuity or simple spans; frame action or pin connections; and he has the choice of free movement or restraint under temperature changes. The final decision as to the nature of the structure is left to the designer, rather than to the inherent characteristics of the material used. For this reason, he is able to suit the structural action to the type of force involved.

But this advantage also carries an added responsibility. The designer must first determine the type of action desired and analyze the structure accordingly. Then, he must design the connections to develop the desired stresses and, most important, these connections must not develop any stresses which are not considered in the analysis. Quite frequently, a connection is used which will do the job intended but, by adding material to this connection, the designer develops forces which have not been included in the analysis.

Two other important factors that must be considered are the available erection and fabrication facilities. A connection that is economical in material may be tedious to erect, or it may demand fabrication precision that will offset savings involved.

The following connections, result from the PCI Connection Details Committee's efforts to show specific details that have proved successful in actual practice. The discussion will describe the purpose of the connection, special design techniques involved, and will show how the connection is consistent with the action it develops. This discussion will be limited to Beam-to-Column connections.

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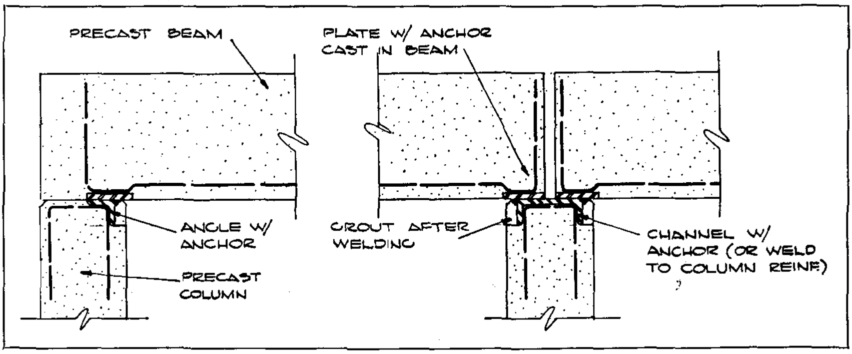


BC (1)—Welded Roof Beam Connection, Simple Spans.

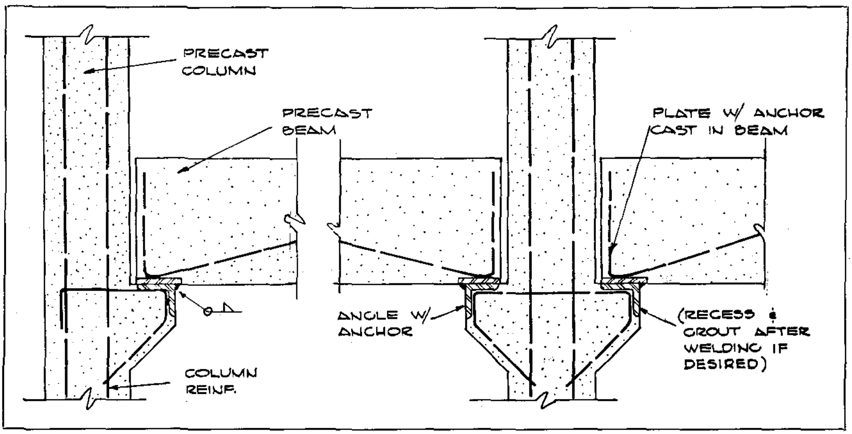
BC 1—This beam to column connection is a good all-weather detail which can be quickly and easily erected, and is usually used where relatively small reactions are involved. The connection as shown makes no provision for beam continuity and, therefore, no connection is made at the top of the beams.

The angle cast in the column can be fireproofed by recessing it into the column as shown, with the recess being grouted after erection.

The interior column uses a channel as a bearing surface, as it is easier to cast into the column than two angles. However, in larger columns, the channel may be replaced with two angles to save material. In this case, the angles should be laced together to prevent splitting the column if the beams shorten due to temperature or shrinkage.

It is essential that the columns and beams have dowels welded to the bearing plates to resist the temperature and shrinkage stresses which develop.

If heavy dead loads are involved, it may be desirable to spot weld the beams to the columns until after erection, and then complete the welding. This will reduce the stresses in the welds.

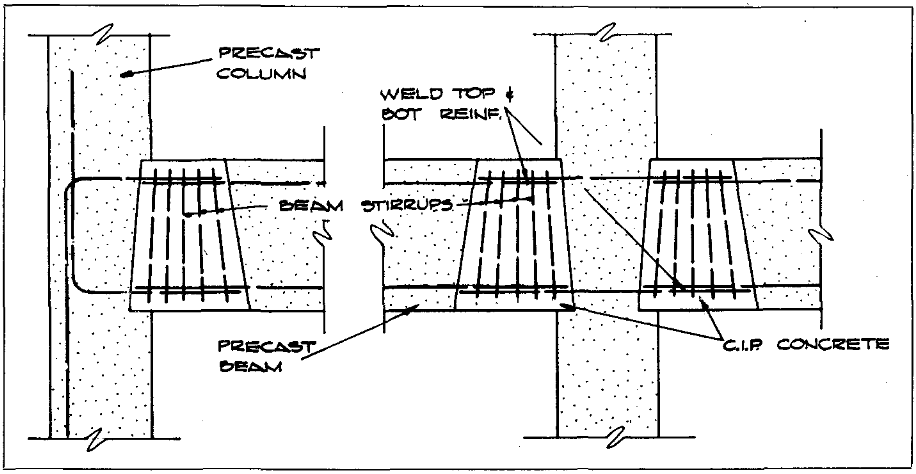


BC (2)—Welded Floor Connection, Simple Spans.

BC 2—This detail is similar to the previous one, but it uses concrete ledges on columns to permit multi-story construction. Notice that no provision is made for continuity and, therefore, the stability of the structure will depend on other means, such as shear walls.

To achieve fireproof construction, the angles in the column haunches may be recessed in the same manner as the previous connection. Here again, the dowels in the beams and columns must be provided because of temperature and shrinkage stresses. It may also be desirable to use a weld procedure similar to the preceeding detail, in the event of heavy dead loads.

In keeping with the concept of providing connections that develop only the desired stresses, the designer should be cautious about using a top connection, as it will tend to develop negative moment which could well crack the beam. If it is necessary to use such a connection, it should be adequate to develop the negative moment that will result.



BC (3)—Cast-in-Place Connection, Continuous Spans.

BC 3—The behavior and appearance of these details are identical with that of a monolithic structure. The beams must be erected on shores until the cast-in-place portion has cured.

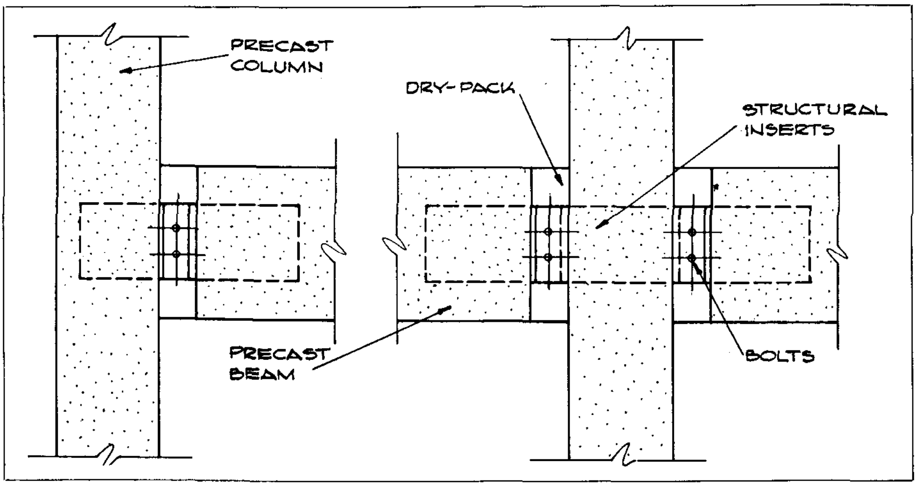
Because the entire beam reaction must be transferred through the cast-in-place concrete to the column, it is desirable to roughen the ends of the beam. The closed ties must be designed as they would be for monolithic concrete.

The beam top steel at the column should be designed to carry the negative moment which will develop and the splices should develop the full tensile capacity of the bars, either by welding or by laps.

BC 4—This connection uses bolts to achieve the initial erection connection and is normally used where light loads are involved.

The structural inserts shown in the beam and column can consist of plates, channels, or other suitable sections. The bolt holes in the inserts are usually slotted for erection tolerance. If possible, high strength bolts in double shear should be used to connect the inserts, and the possibility of eccentric load due to unequal spans or loadings should be considered.

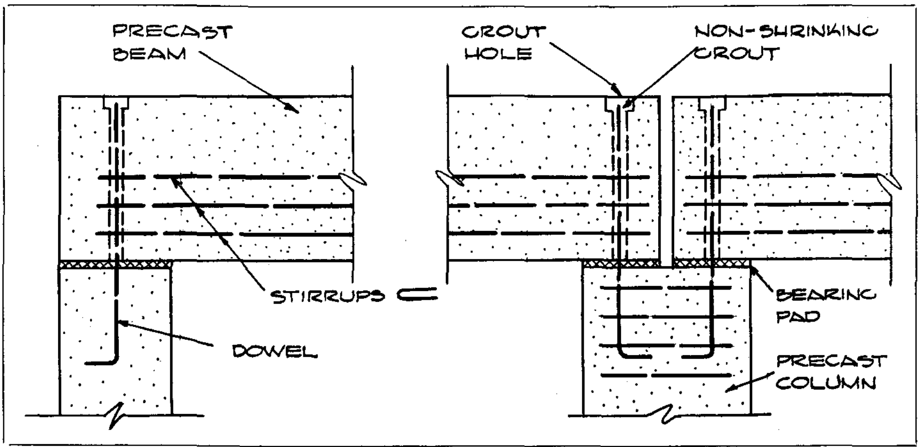
In this connection, several possible modes of failure exist. Bolt shear, plate bending, the bearing stresses of the plate on the concrete, and torsion, should be investigated. It is usually necessary to weld dowels to the inserts



BC (4)—Bolted Connection, Simple Spans.

to achieve a satisfactory load transfer from the beam to the column.

The space between the ends of the beam and the face of the column is filled with grout to protect the inserts. By roughening the concrete surfaces, the grouted area can assist in transferring the beam shear to the column.

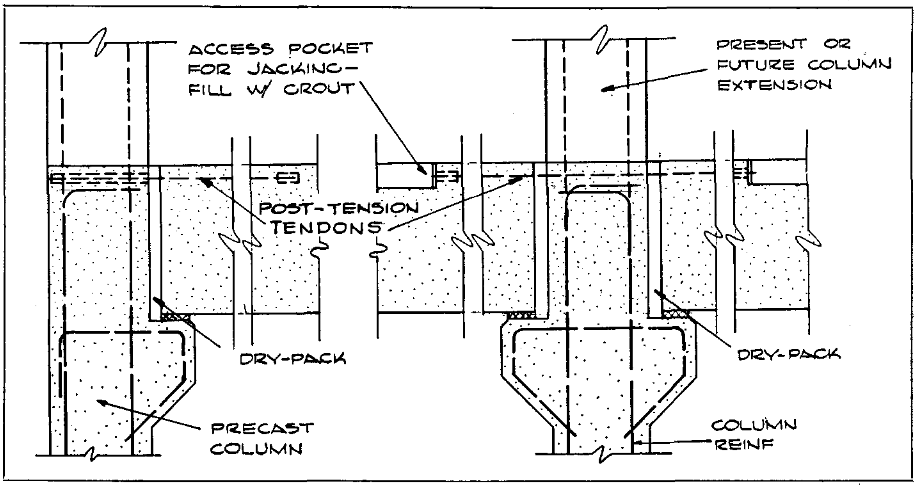


BC (6)—Dowelled Roof Beam Connection, Simple Spans.

BC 6—This connection sketch shows a roof condition. In this detail, the sleeves in the beams should be large enough to provide for fabrication and erection tolerances.

In the event long spans are involved, the rotation at the ends of the beams may be sufficient to damage the roofing. If this is the case, the roofing should be detailed to accommodate this movement or, perhaps, connections which develop continuity should be considered.

BC 7—This connection uses post tensioned tendons to develop full negative moment across the support. The tendons may be cast in the beam or, most commonly, they are installed after erection. If the tendons are inserted in the field, through sleeves cast in the beams and columns, the space be-



BC (7)—Post-tensioned Connection, Continuous Spans.

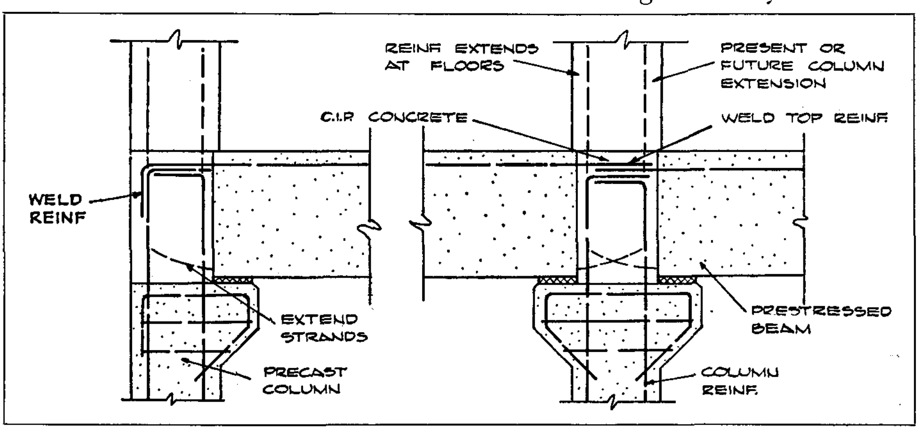
tween the tendons and the sleeve should be grouted to prevent deterioration.

It is essential that the joint between the beam and the column be packed with grout, or else the full tension force of the units will be transferred to the beam as direct tension. This could cause the beam to crack severely at the end of the unit. Particular attention should be payed the stresses developing around the access pocket and reinforcement should be provided.

An advantage of this connection is that when used with a prestressed beam, it provides a monolithic, uncracked frame in both the positive and negative regions. This results in a saving in shear reinforcement and a reduction in the deflection.

The tendons will naturally be relatively short and, therefore, a small change in their length causes an appreciable change in stress. For this reason, seating of the anchorages becomes particularly important, and careful study should be given to the devices used. It is usually desirable to have an engineering representative of the tendon supplier supervise the tensioning until the erection crews are thoroughly familiar with the procedure.

BC 8—This detail shows another method of achieving continuity and mono-



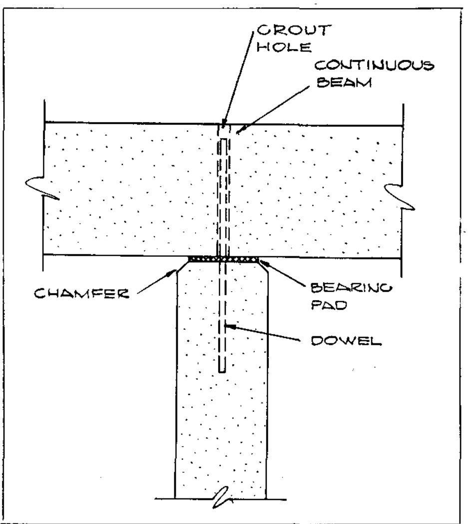
BC (8)—Welded Connection, Continuous Spans.

lithic behavior between two adjacent beam spans. It also lends itself readily to future expansion, as the space between the beam ends is sufficient for extending the column steel.

The top reinforcing, which must be sufficient to develop the negative moment, may be spliced either by welding or merely by lapping the bars sufficiently.

If the beam is prestressed, the strand may be extended into the cast-in-place portion to develop temperature, shrinkage or seismic forces. However, a more satisfactory solution is to extend dowels from the end of the beam. These dowels extend back into the beam and lap with the strand.

In this joint, as in a monolithic joint, the column should be designed for its portion of the frame moment, particularly where the possibility of unsymetrical loading occurs.



BC (9)—Dowelled Connection, Continuous Spans.

BC 9—The detail shown here indicates a method of achieving a beam to column connection where the beam bears across the column. This condition occurs in the case of beam cantilevers, or at an interior column where the beam is more than two spans long. It may also occur at an exterior column.

This sketch shows a beam, continuous over the column, with a doweled connection to the column. If the beam is not a monolithic part of the column, the edges of the column should be chamfered and a bearing pad provided to prevent spalling of the concrete.

The dowels are normally cast into the column, but they may be grouted into oversized holes, if a particular fabrication or erection condition exists. The sleeve through the beam should be large enough to provide for production and erection tolerances, and the beam width should reflect the area of concrete removed by the sleeve.

As in previous doweled connections, a bolt may be used if an immediate erection connection is required. In the event bolts are used, the sleeve may be left ungrouted, unless the bolt is subject to weathering.

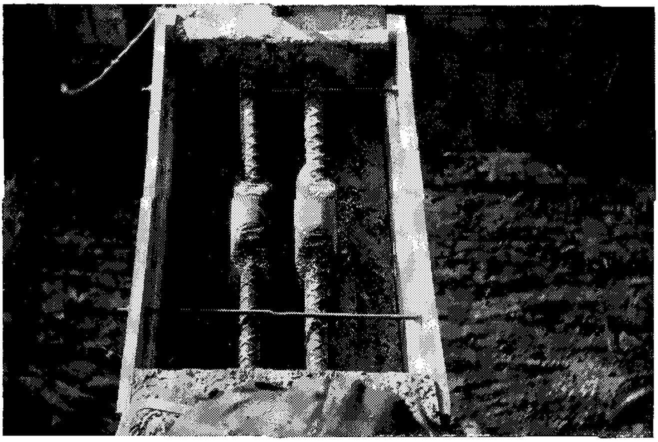


Fig. 1—Beam to column connection BC-8.

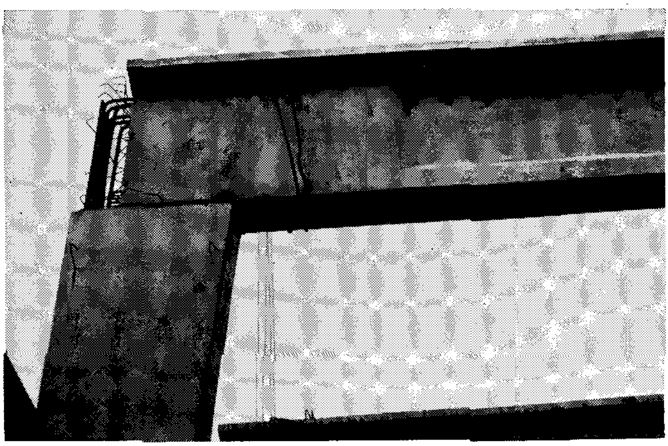


Fig. 2—Beam to column connection BC-8.

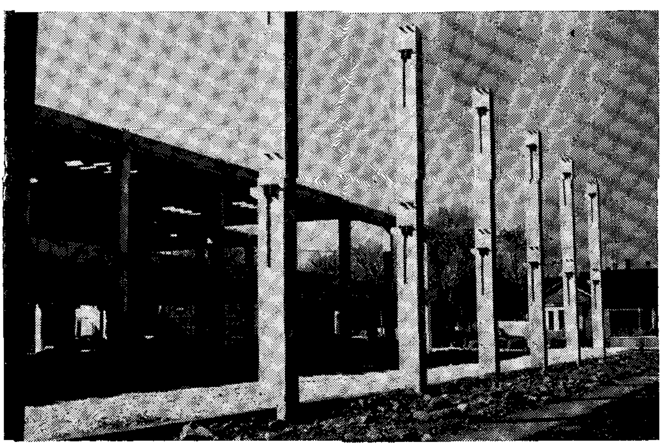
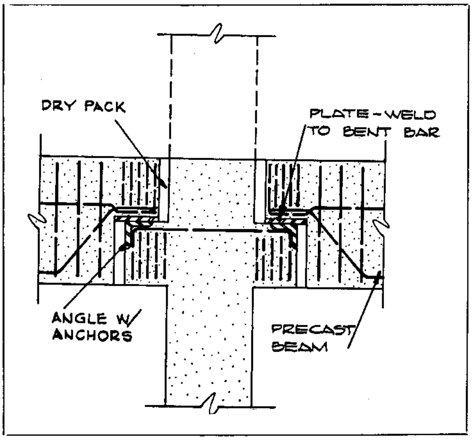


Fig. 3—Vertical plate haunches BC(c).

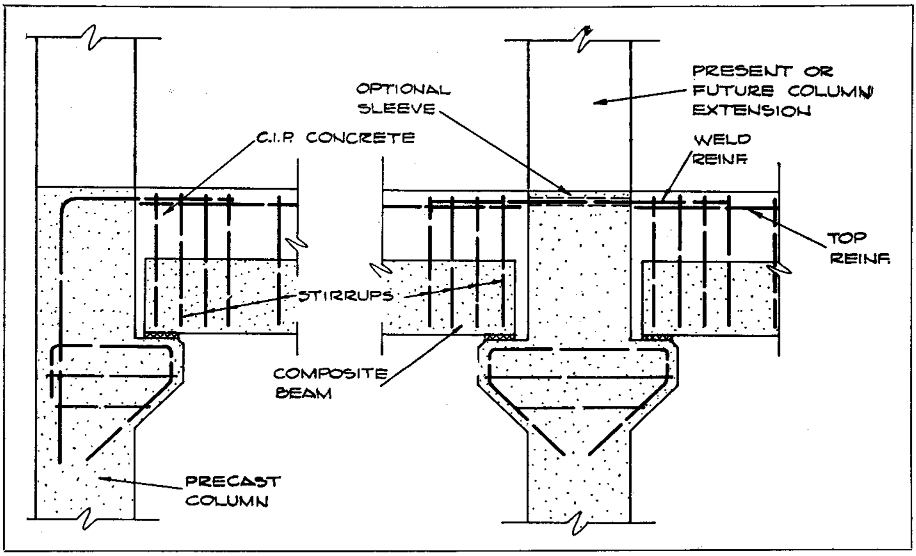


BC (10)—Welded Connections, Simple and Continuous Spans.

BC 10—This detail uses concrete haunches, cast on the column, to support the beam reactions. However, these haunches are recessed for appearance, and to reduce the depth of construction.

The connection is, naturally, limited in shear capacity by the notched beam, and does not provide for continuity. However, it is not necessary to shore the beams. The reinforcement in the notched portion of the beam is particularly important and it must be sized to resist shear, temperature, and shrinkage stresses based on the reduced depth. In addition, study should be given to the secondary moment developed by the column haunch, and the beam seat, acting as short cantilevers.

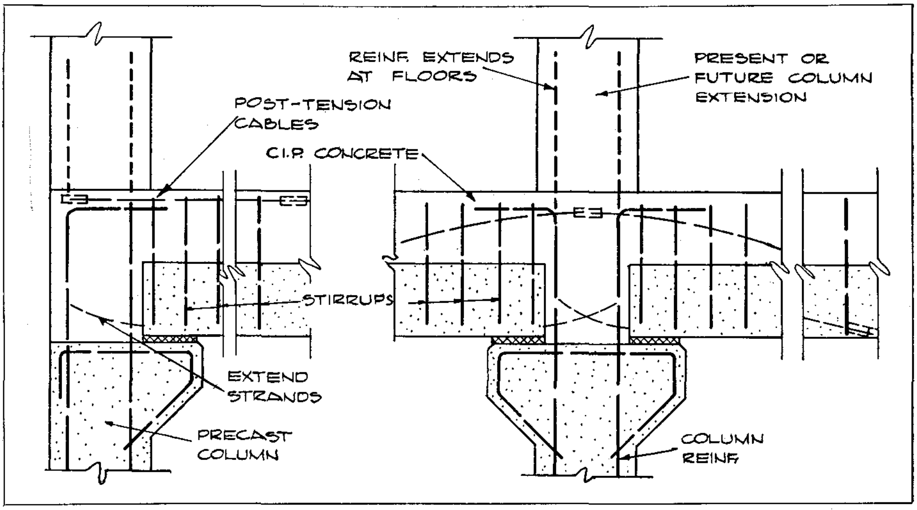
In this connection, the diagonal bars should be designed to resist the total shear in order to be sure of load transfer across the joint.



BC (11)—Composite Beams, with Welded or Lap Spliced Reinforcement, Continuous Spans.

BC 11—The connection shown in this sketch is used primarily with precast soffit beams supporting precast slabs. The soffit beam is erected on the column ledge and shored, if necessary. The cast-in-place portion above the soffit is used to develop composite action for the beam. The connections at both the exterior and interior columns behave as monolithic joints. The steel in the top of the composite portion at the columns must be designed for the negative moment which occurs. The magnitude of this moment will depend on the shoring procedure. The negative moment bars may be cast integrally with the columns or placed through sleeves. The splices may be either welded or lapped.

The stirrups projecting from the soffit should be sufficient to develop the horizontal shear between the composite portion and the soffit. As in all concrete-to-concrete bearing surfaces, steel plates or bearing pads should be used between the column ledges and the beam. It is also necessary to insure that the space between the end of the soffit and the face of the column be filled with concrete, to transfer the compressive stresses due to negative bending. The reinforcing in the column haunches should be adequate to develop the shear and moment caused by the beam reaction.



BC (12)—Composite Beams, Post-tensioned, Continuous Spans.

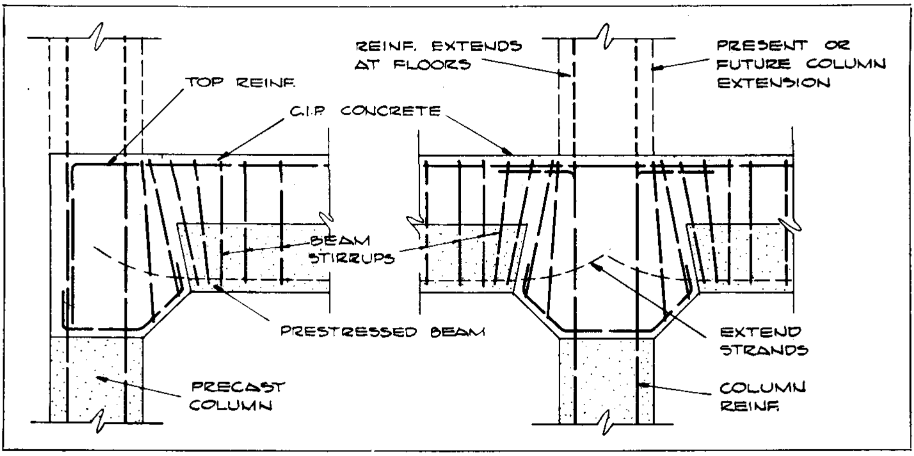
BC 12—This connection is a modification of the previous one, with post tensioning units used to develop the negative moment.

As these units are placed in the cast-in-place concrete, the necessity for sleeves is eliminated, except where the tendons pass through the soffit.

The particular problems discussed previously concerning the use of short prestressing elements are equally as important in this connection.

BC 13—This connection is a third variation of the precast soffit beam and composite concrete type of construction. The increased depth of construction at the column provides additional shear and moment capacity, provided the additional depth extends far enough into the span to be of significant value.

An advantage of the sloped bottom edge of the shear head is that it allows for relatively large tolerances in the length of the precast members

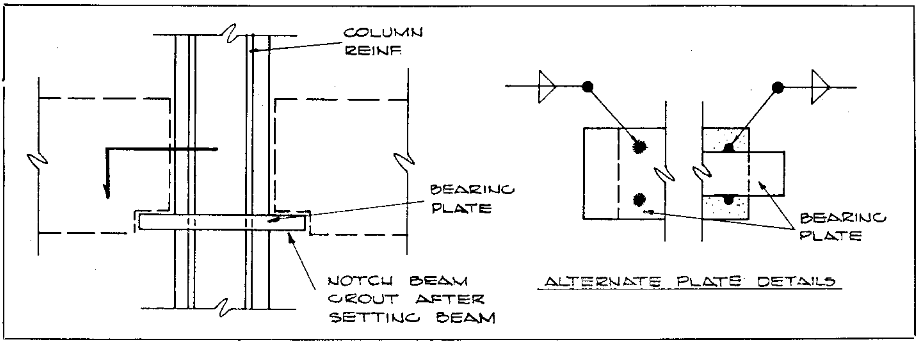


BC (13)—Composite Beams with Cast-in-Place Haunches, Continuous Spans.

without altering the appearance.

The design of the reinforcement for negative moment, horizontal and vertical shear, and temperature and shrinkage stresses is identical with the previous soffit construction.

It is possible to design these continuous types of connections in a manner somewhat different from normal elastic design. The negative moment reinforcement over the column may be designed for less than the full moment indicated by an elastic analysis, provided the positive moment capacity is sufficient to allow for the plastic condition which will occur. However, in no case should the continuity steel be less than that required to resist ⅔ of the elastic negative moment. Particular attention should be given to the more severe cracking that may occur at the column due to increased elongation of the steel. This can result in a more severe shear problem.



BC (a)—Horizontal Plate Haunch.

BC (a)-Quite a few of the details discussed so far have used concrete haunches on the precast columns. Some engineers and architects consider this an absolute last resort.

This detail, and the next two, show methods of developing steel haunches to support beam reactions. They are not shown as connections, only as haunch types.

This detail uses a flat steel plate cast in the column. Depending on the load involved, the plate may be the same width as the column, or narrower.

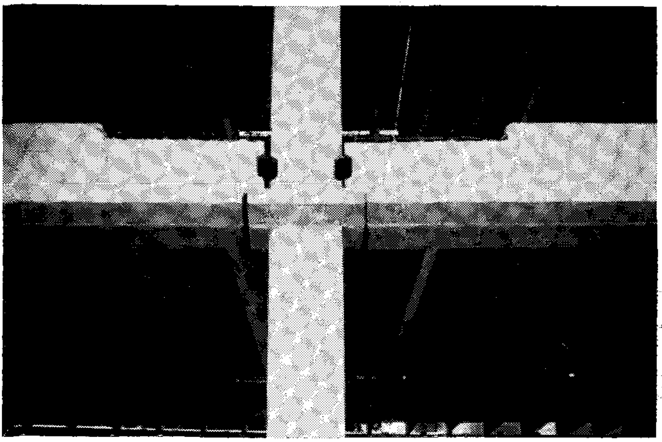


Fig. 4—Beam to column connection BC-10.

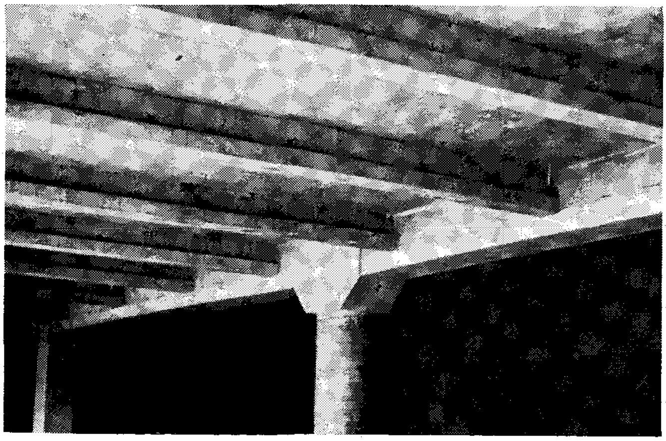


Fig. 5—Beam to column connection BC-13.

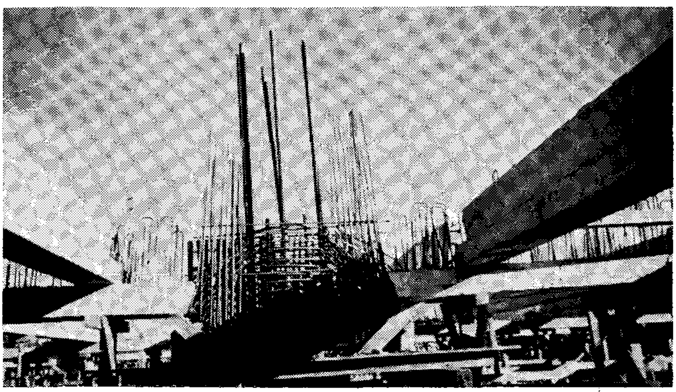
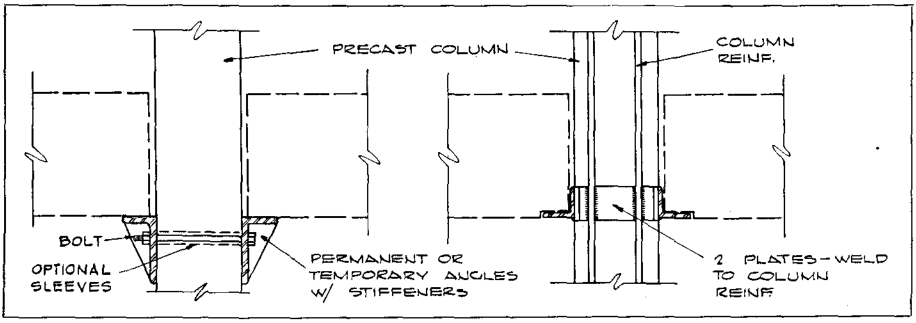


Fig. 6—Beam to column connection BC-13 under construction.

If the plate is full width, the column reinforcing must pass through it. If the plate is narrower than the column, the bars may pass on either side of it. In either case, the plate should be welded to the bars to insure proper load transfer to the columns.

The plate itself should be designed to cantilever from the column bars, not from the edge of the column.

The plate may be flush with the bottom of the beam, or recessed, if concrete cover is necessary for fire protection or appearance.



BC (b)—Bolted Angle Haunch.

BC (b)-These sketches show additional methods of developing a steel haunch. The detail on the left uses loose angles to support the beams. These angles are bolted to the column by the use of loose bolts, which pass through the sleeves cast in the column. The same connection could be made by using fixed bolts cast into the column, although this would necessitate cutting the column form. In designing this connection, care should be taken to control the concrete bearing stresses below the bolts, as well as bending in the angle legs. The bolts should also be far enough above the lower toe of the angle to prevent it from twisting off the column.

One of the main advantages of this detail over the previous one, which used the flat plate, is the savings in material. The flat plate will normally be rather thick, in order to offer the necessary section modulus.

Stiffeners may be used between the angle legs to reduce the thickness required. This bolted angle connection is frequently used as a temporary erection connection, when the final load transfer is to be accomplished by other means.

The detail on the right is another method for using angles to support the beam reaction. In this case, the angles are recessed into the beam for fireproofing or appearance. The angles are welded to the column plates, either in the field or in in the precasting yard.

The column unit consists of face plates welded to flat bars, which are in turn welded to the column reinforcing. With the angles recessed in this manner, it is more difficult to stiffen the angles. The most common way this is done is to use two stiffeners, one on either side of the beam. An alternate method is to cast a stiffened angle into the toe of the beam, and then weld the beam to the seat angle, so that both angles act as a unit.

BC (c)-This sketch shows a third type of steel haunch for beam support. The connection is normally used where heavier beam reactions are involved.

Fig. 7—Horizontal plate haunch BC(a).

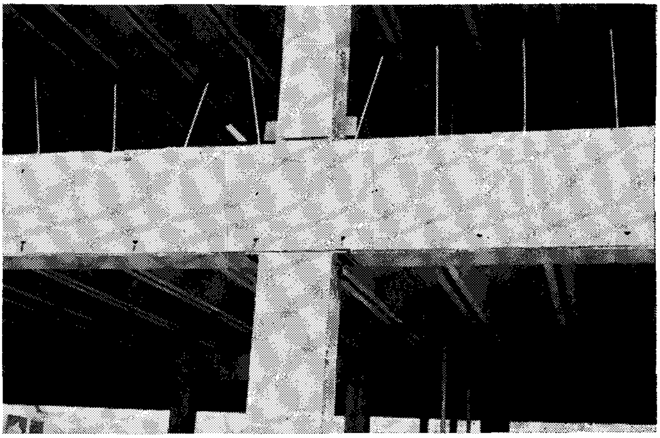


Fig. 8—Horizontal plate haunch BC(a).

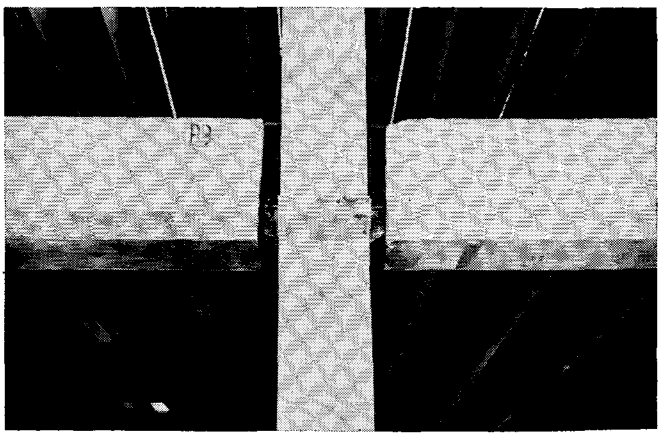
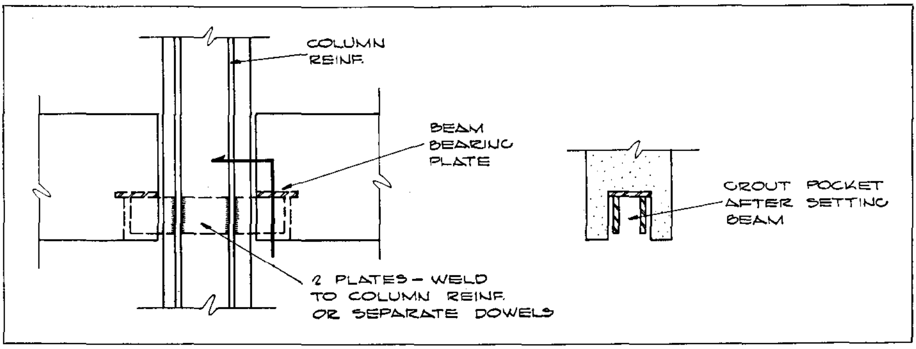


Fig. 9—Vertical plate haunch BC(c).



BC(c)—Vertical Plate Haunch.

The seat itself consists of two flat bars, cast vertically in the column. The vertical plates offer a great deal more section modulus than a flat plate or angle, and therefore are capable of taking larger reactions.

These bars are also welded to the column reinforcing and again the cantilever moment is calculated at the column bars, rather than at the face of the column. The bars should be spaced far enough apart to provide stability to the beam during erection, and the plate in the beam should be large enough to control the concrete bearing stresses.

It is also desirable to only use two support bars, as it is difficult to achieve equal bearing on more than two.

This connection offers excellent appearance and fireproofing characteristics, when it is recessed as shown.

CONCLUSION

There is one pitfall in the design of a precast structure that has trapped every designer at some time. Each of the details we have described have been finished connections, and our discussion has described the analysis of the joint as it will act in the finished structure. But it is equally important to study the structural action during the erection stage. This is particularly critical in multi-story work.

For example, suppose we consider a multi-story building that is to be made fully continuous under service loads. The columns will be designed on an unsupported length of only one story because there is frame action with the beams.

However, this building may be erected to full height before the cast-in-place continuity connections are made. With a heavy dead load, the stability of the structure could well depend on temporary erection connections.

This example of erection stability is only one of the many special design considerations a precast frame requires of the designer. However, I think we have also shown the enormous versatility offered by precast concrete.

As long as designers, fabricators, and erectors are fully aware of the action developed in the structure they are dealing with, as well as the limitations and advantages of any particular system, we will see strides in the industry even greater than have been made in the past decade.