Second, it is suggested that in high rise composite frame construction that the vertical bar splices be located at the middle clear height of the composite column. This point is usually near the inflection point (zero moment) of the column where the more economical compression lap splices or compression butt splices may be used instead of the more expensive tension lap or tension butt splices that may be required if splices were made at the floor line.

A suggested composite column splice detail is shown in Fig. 12.

Steel Beam to Embedded Wide Flange Connection

The use of composite columns in composite frame construction often utilizes steel spandrel and/or perpendicular floor beams framing into the column at the floor level. Sometimes these beams will be simply supported floor beams where conventional double angle framed beam connections (PART 5 - LRFD Manual) or single plate shear connections may be utilized. More often, however, the steel spandrel beams will be part of the lateral load resisting system of the building and require a moment connection to the composite column. Practicality will often dictate that the larger spandrel beam be continuous through the joint with the smaller erection column interrupted and penetration welded to the flanges of the spandrel beam. To increase the speed of erection and minimize field welding, the spandrel beam and erection column are often prefabricated in the shop to form 'tree columns' or 'tree beams' with field connections at midheight of column and midspan of spandrel beam using high strength bolts.

The engineer must concern himself with the transfer of forces from the floor beams to the composite column. For simply supported beams not part of the lateral frame, the simplest method to transfer the beam reaction to the composite column is through a standard double angle or single shear plate connection to the erection column. It is then necessary to provide a positive shear connection from the erection column to the concrete along the column length to ensure transfer of the beam reaction to the composite column cross-section. The simplest method to accomplish this is by the use of standard headed shear connectors preferably shop welded to the erection column. For moment connected spandrel beams, the beam shear and unbalanced moment must be transferred to the composite column cross-section. Different transfer mechanisms have been tested at the University of Texas at Austin[16,17]. One suggested detail is shown in Fig. 14.

Shear Connectors

As discussed in the previous section, it is necessary to provide a positive shear connection transfer from the floor beam to the embedded steel column when the beam connection is made directly to the embedded steel column. It is likely that a significant portion of this reaction can be transferred in bond between the embedded section and the concrete as reported in Ref. 14. An estimate of this value can be made from eqn (5) of Ref. 14 which is based on the results of a limited number of push tests in which a steel column is embedded in a concrete column.

$$P = \frac{3.6 * b_f \; (0.09 * f'c - 95) * l_c}{k}$$

where

P = service load capacity on the embedded shape,
b_f = steel flange width of embedded shape,
f_c = concrete compressive strength (PSI),
l_e = embedded length of steel shape,
k = constant, 5.

Converting to an average Ultimate bond stress *u* using only the flange surfaces as being effective and applying a safety factor of 5 as reported in the tests,

$$u = \frac{P \times 5}{4 b_f l_e} = \frac{3.6 \times b_f (0.09 f'c - 95) l_e}{5} \times \frac{5}{4 b_f l_e}$$

$$u = 0.9(0.09 f'c - 95),$$

average ultimate bond stress in PSI (1)

If this equation is applied to a typical case of a W14 x 90 embedded column in 5000 PSI (34 500 kPa) concrete with a floor to floor height (*h*) of 13 ft (3.96 m),

$$u = 0.9\ (0.09 \times 5000 - 95) = 320 \text{ psi } (2206 \text{ kPa})$$

available shear transfer = $u \times h \times 4b_f$

$$\text{ultimate load, kips} = \frac{320 \times (13 \times 12) \times (4 \times 14.5)}{1000}$$

$$= 2895^k\ (12900 \text{ kN}) \qquad (2)$$

These results indicate that all of any typical floor reaction to the composite column could be easily transferred to the concrete in bond alone.

The above discussion considered the case where axial load alone is transferred from the embedded steel section to the concrete. For beam columns where high bending moments may exist on the composite column, the need for shear connectors must also be evaluated. Until such time as research data are provided, the following simplistic evaluation may be made. Assume a situation where a composite column is part of a lateral load resisting frame with a point of inflection at mid-column height with a plastic neutral axis completely outside the steel cross-section (Fig. 15). An analogy can be made between this case and that of a composite beam where shear connectors are provided uniformly across the member length between the point of zero moment and maximum moment. The ultimate axial force to be transferred between the embedded steel column and the concrete over the full column height is $2AFy$ where A is the steel column area and Fy is its yield strength. Assuming a bond strength is available in this case similar to the case of the push test eqn (1), then shear connectors would theoretically be required when *2AFy* is greater than the results of eqn (2). Taking as an example, a W14 x 90 erection column

$$2AFy = 2 \times 26.5 \times 36 = 1908^k \ (8486 \text{ kN})$$

Available shear transfer from bond (2) is,

$$4uhb_f = \frac{4 \times 320 \times (13 \times 12) \times 14.5}{1000}$$

$$= 2895^k \ (12900 \text{ kN}) \rangle\ 1908^k \ (8486 \text{ kN})$$

Again, it is shown that bond stress alone can transfer the shear between the embedded shape and the concrete, assuming no loss in bond as a result of tensile cracking present at high moments. The LRFD specification commentary in Section I.4. discusses design using the plastic stress distribution of the full composite cross-section and requires a transfer of shear from the steel to the concrete with shear connectors.

Until further research is conducted on the loss of bond between the embedded steel section and the concrete and more comprehensive push tests are run, the following suggestions are made with regard to shear connectors on composite columns:

1. Provide shear connectors on the outside flanges where space permits. Where space does not permit provide shear connectors on the inside flange staggered either side of the web.

2. Provide shear connectors in sufficient quantity, spaced uniformly along the embedded column length and around the column cross-section between floors to carry the greater of the following minimum shear transfer forces as applicable:

   a. the sum of all beam reactions at the floor level;
   b. whenever $P_u/\phi_c P_n$ is less than 0.3, a force equal to Fy times the area of steel on the tensile side of the plastic neutral axis in order to sustain a moment equal to the nominal flexural strength of the composite-cross section. 0.3 is used as an arbitrary point separating a composite column subjected to predominantly axial load and one subjected to predominately moment. Consideration must be given to the fact that this moment is reversible.

3. The maximum spacing of shear connectors on each flange should be 813 mm.

If minimum shear connectors are provided according to the guidelines identified herein, it is reasonable to assume compatibility of strains between concrete and embedded steel to permit higher strains than 0.0018 under axial load alone. This strain level has been identified in Ref. 18 and the LRFD commentary Section I.2.1 as the point when unconfined concrete remains unspalled and stable. Therefore, a slight increase in the allowable reinforcing steel stress from 380

Mpa corresponding to 0.0018 axial strain to 415 MPa would seem to be justified. The use of shear connectors also allows the full plastic moment capacity to be counted upon when $P_u/(\phi_c M_n)$ is less than 0.3 (LRFD Commentary I.4) instead of the reduction specified in LRFD Specification Section I.4.

Suggested details for shear connectors on composite columns are shown in Figs. I2 and 13.

Base Plates

Normally a base plate for the embedded steel column of a composite column is specified to be the minimum dimension possible to accommodate the bolts and anchoring it to the foundation during the erection phase and to carry the erection loads. In so doing, the base plate will interfere the least possible amount with dowels coming up from the foundation to splice with the longitudinal vertical bars of the composite column. The design engineer must remember that any area of base plate assume to transmit axial load to the foundation from embedded steel column must not also be used to transmit concrete bearing stresses to the foundation from the concrete portion of the composite column. It may be necessary, depending on the size of the steel column and number of shear on it, to add additional foundation dowels to adequately transmit the foundation load carried by the concrete of the composite column.

COMPOSITE COLUMN DESIGN BY LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

In order to qualify as a composite column under the LRFD specification design procedure, the following limitations must be satisfied as defined in Section I.2.1:

1. The cross-sectional area of the steel shape, pipe or tubing must comprise at least 4% of the total composite cross-section.

2. Concrete encasement of a steel core shall be reinforced with longitudinal load carrying bars, longitudinal bars to restrain concrete and lateral ties. Longitudinal load carrying bars shall be continuous at trained levels; longitudinal restraining bars may be interrupted at framed levels. The spacing of ties shall be not greater than 2/3 of the least dimension of the composite cross-section. The cross-sectional area of the transverse and longitudinal reinforcement shall be at least 0.007 in^2/in (0.178mm^2/mm) of bar spacing. The encasement shall provide at least I.5 in (38 mm) of clear cover outside of both transverse and longitudinal reinforcement.

3. Concrete shall have a specified compressive strength f'c of not less than 3 ksi (20 700 kPa) nor more than 8 ksi (55 100 kPa) for normal weight concrete and not less than 4 ksi (27 600 kPa) for lightweight concrete.

4. The specified minimum yield stress of structural steel and reinforcing bars used in calculating the strength of a composite column shall not exceed 55 ksi (379 000 kPa).

f_y calc $\leq$ 55 ks