A Tentative Design Guideline for a New Steel Beam Connection Detail to Composite Tube Columns

ATOROD AZIZINAMINI and BANGALORE PRAKASH

Steel tubes of relatively thin wall thickness filled with high-strength concrete have been used in building construction in the U.S. and Far East Asian countries. This structural system allows the designer to maintain manageable column sizes while obtaining increased stiffness and ductility for wind and seismic loads. Column shapes can take the form of tubes or pipes as required by architectural restrictions. Additionally, shop fabrication of steel shapes helps insure quality control.

In this type of construction, in general, at each floor level heavy steel beam is framed to these composite columns. Often, these connections are required to develop shear yield and plastic moment capacity of the beam simultaneously.

This paper summarizes results and recommendations from a pilot study conducted to develop a moment-resisting steel connection detail for connecting steel beams to composite columns of the type described above. The focus of this pilot study was on composite columns having a square or rectangular cross section.

CURRENT PRACTICE

Beam-column connections in concrete-filled steel tubes are usually constructed by directly welding the steel beam to the tube when connections are required to develop plastic moment capacity of the beam. Current design practices for these connections rely heavily on the judgment and experience of individual designers, with little research and testing information available.

When beams are welded or attached to steel tubes through connection elements, complicated stiffener assemblies are required in the joint area within the column. However, welding of the steel beam or connecting element directly to the steel tube of composite columns should be avoided for the following reasons:

1. Transfer of tensile forces to the steel tube can result in separation of the tube from the concrete core, thereby overstressing the steel tube. In addition, the deformation of the steel tube will increase connection rotation, decreasing its stiffness.
2. Welding of the thin steel tube results in large residual stresses because of the restraint provided by other connection elements.
3. The steel tube is designed primarily to provide lateral confinement for the concrete which could be compromised by the additional stress due to the welded connection.

Atorod Azizinamini is an assistant professor, Department of Civil Engineering, University of Nebraska-Lincoln, Lincoln, NE.

Bangalore Prakash, structural engineer with Nabih Youssef and Associates, Los Angeles, California, formerly a graduate student, Department of Civil Engineering, University of Nebraska, Lincoln, NE.

POSSIBLE CONNECTION DETAIL

With these considerations in mind, attempts should be made to prevent direct transfer of beam forces to the steel tube. Two general types of connection details were envisioned, types A and B.

Type A Connection Detail

Figure 1 shows one alternative in which forces are transmitted to the core concrete via anchor bolts connecting the steel elements to the steel tube. In this alternative, all elements could be pre-connected to the steel tube in the shop. The nut inside the steel tube is designed to accomplish this task. The capacity of this type of connection would be limited with the pull-out capacity of the anchor bolts and local capacity of the tube.

Another variation of the same idea is shown in Figure 2, where connecting elements would be embedded in the core concrete via slots cut in the steel tube. In this variation slots must be welded to connection elements after beam assembly for concrete confinement. The ultimate capacity of this detail also would be limited to the pull-out capacity of the connection elements and the concrete in the tube.

Type B Connection Detail

Another option is to pass the beam completely through the column (see Figure 3). This type of connection is believed to be the most suitable. In this type of detail a certain height of column tube, together with a short beam stub passing through the column and welded to the tube, could be shop fabricated to form a "tree column." The beam portion of the "tree column" could then be bolted to girders in the field. A combination of analytical and experimental investigations was undertaken to comprehend and identify the force transfer mechanism and suggest a tentative design procedure for this type of connection.

ANALYTICAL INVESTIGATION

To investigate the performance of the connection detail in which the beam completely passes through the column (hereafter referred to as a through connection), detailed finite element analyses were conducted. The finite element model used in these analyses consisted of a three-dimensional model of the column with a small portion of the beam extending from the column. In these analyses concrete cracking and non-linear behavior of the steel elements were modeled. In addition, the interface between steel and concrete elements was carefully modeled.

Results of the analyses were used to identify the force transfer mechanism between the steel beam and composite column in the joint region, and to identify the effects of some of the connection details on its performance. Major conclusions from the analytical investigation associated with the through beam connection detail are discussed in the following section.

Figure 4 shows the force transfer mechanism observed from the analyses. The portion of the steel tube between the beam flanges acts as a stiffener, resulting in a concrete compression strut which assists the beam web within the joint in carrying shear. The effectiveness of the compression strut was shown to be increased to a limit by increasing the thickness of the steel plate between the beam flanges. The width of the concrete compression strut on each side of the beam web in the direction normal to the beam web is approximately equal to half the beam flange width.

A compressive force block is created when beam flanges are compressed against the upper and lower columns (Figure 4). The width of this compression block is approximately equal to the width of the beam flange. In the upper and lower columns shown in Figure 4 the compressive force, C, is shown to be balanced by the tensile force provided by an embedded rod in the concrete and possibly welded to the beam flanges. This rod was not modeled in the finite element model, forcing the steel liner plate to carry this tensile force.

Since one of the objectives of this phase of the study is to devise means to improve connection performance, it is beneficial to require rods be attached to beam flanges as shown in Figure 4. The presence of such rods is believed to make the beam web within the joints stiffer and reduce the stress level in the steel tube.

EXPERIMENTAL INVESTIGATION

To gain additional insight of the behavior of the through beam connection detail, one test specimen representing approximately a one-half scale model of a prototype column used in high-rise building construction in seismic zones was con-

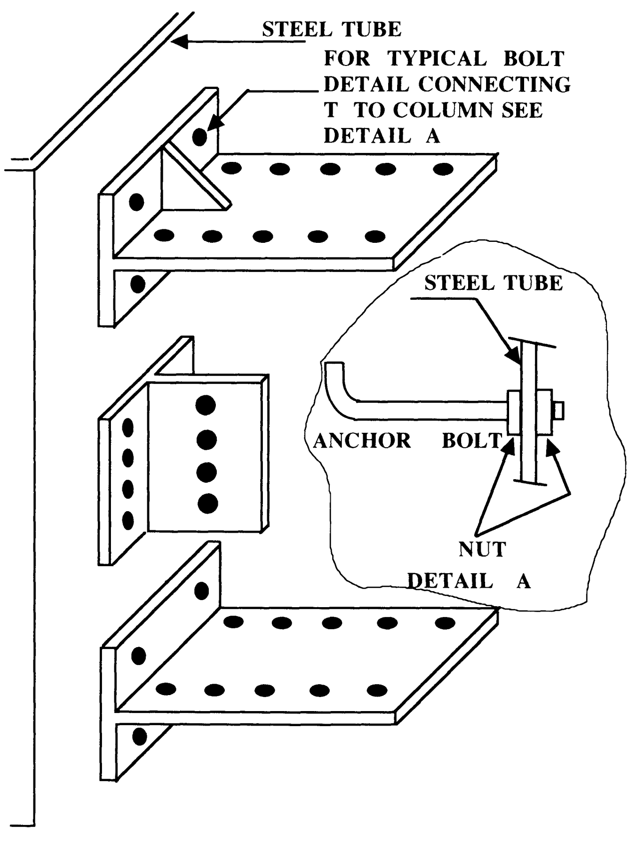

Fig. 1. Type A connection detail using anchor bolt.

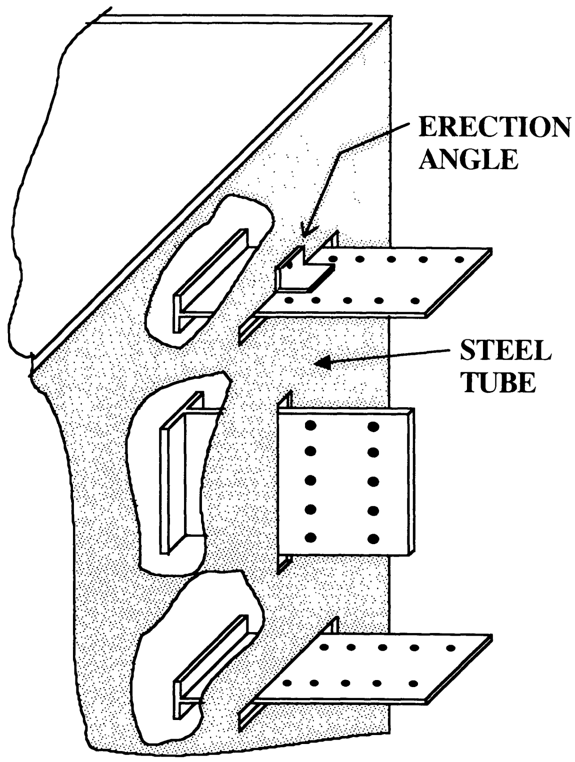

Fig. 2. Type A connection detail using embedded elements.

structed and tested. The prototype column consists of a 4-ft (1.22-m) square hollow tube with a 2-in. (50.8-mm) wall thickness, 10′-9 (3.28-m) story height, and W30x99 beam section framing to the column. In this particular building the W30x99 beams were welded directly to the steel tube. To prevent overstressing of the steel tube a complicated scheme of stiffener assemblies was placed inside the hollow tube directly behind the beam section.

Figure 5 shows the general configuration of the test specimen. The height of the column from the beam's top and bottom flanges to the support point is $31\frac{11}{16}$ inches (0.8 m) and represents the distance from the floor to the inflection point in the upper and lower stories of a building frame subjected to lateral loading (assuming the inflection point to be located at mid-height of the column). The length of the beam extending from each side of the column is 27 inches (0.69 m). This length was selected such that the beam's cross-section shear yield and plastic moment capacities would develop simultaneously.

Figure 6 shows the different components of the test specimen. The test specimen consisted of three major components:

a. hollow steel tube made of A36 steel
b. hybrid built-up beam section
c. four #11 grade 60 reinforcing bars with anchor plates welded to each end of the reinforcing bars

The hollow steel tube is 24 inches (0.6 m) square with ½-in. (12.7-mm) wall thickness. A half-scale model of the prototype column (which has a 2-in. (50.8-mm) wall thickness) would have required using 1-in. (25.4-mm) wall thickness in the test specimen. However, only ½-in. (12.7-mm) wall thickness is used.

As shown in Figure 6, two slots in the shape of the beam cross section were prepared on two faces of the steel tube. These slots were used to pass the beam through the column.

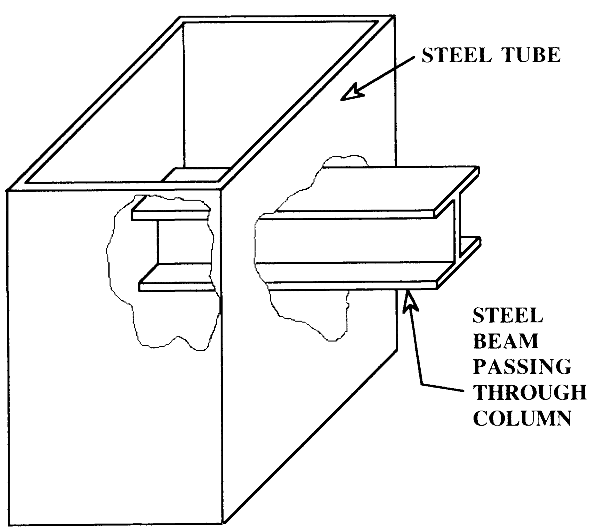

Fig. 3. Through connection detail.

Four holes were drilled on each flange of the beam within the column as shown in Figure 6. These holes were used to pass four #11 grade 60 (414 MPa) reinforcing bars through the beam flanges. Reinforcing bars were then welded to the beam flanges. As discussed earlier, these reinforcing bars were provided to resist tensile forces in the lower and upper columns arising from applied beam loads. The 4×2×l-in. (102×50.8×25.4-mm) plates welded to each end of the reinforcing rods were intended to reduce the amount of slip in the rebars. "Excessive" slip of the rebars could transfer large tensile forces to the steel tube. It may be possible to achieve this same objective by using longer rebars (develop the rebars) or by using a hook at the end of the rebars, particularly since it has been reported that the use of steel plates at the end of anchor bolts could potentially reduce their capacity.[1]

The specimen was cast and cured in the vertical position. The concrete compressive strength at time of testing was 14,000 psi (99 MPa).

TEST RESULTS

In this section the general behavior of the test specimen in terms of function of the beam web within the joint are described briefly. Further details are given elsewhere.[2]

Figure 7 shows the location and orientation of six gages attached to the beam web within the column. Also shown in this figure is the direction of the applied beam loads. Data from these gages, as shown in Figure 8, indicate that the beam web within the joint is subjected primarily to compressive and tensile strains along the lines GG and HH, respectively. This

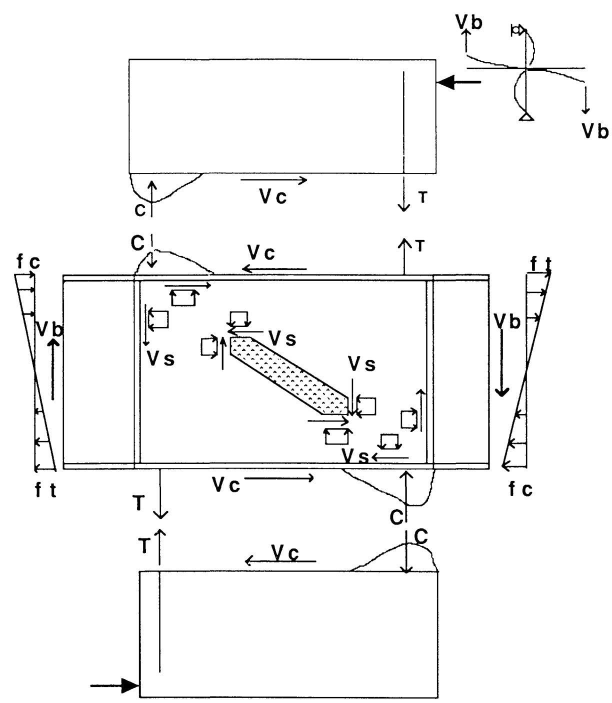

Fig. 4. Force transfer mechanism for through beam connection detail.

type of deformation indicates that the beam web experiences shear type deformation.

Closer examination of data from gages shown in Figure 7 indicate that tensile strains along lines parallel to HH are significantly larger than compressive strains parallel to line GG. This observation can be explained as follows. The type of shear deformation imposed on the beam web within the joint results in the creation of a concrete compressive strut parallel to line GG in Figure 7. This compressive strut acts as a stiffener along the diagonal GG, consequently reducing the compressive strain in the beam web in that direction. However, in the other direction (along line HH) tensile strains in the web increase since concrete is not effective. This observation verified the force transfer mechanism deduced from the analytical investigation and explained earlier.

BEHAVIORAL MODEL

Based on results of the finite-element analysis and experimental results, a behavioral model in the form of equations relating the applied external forces to the connection's internal forces was developed. These equations are then used to suggest a tentative design criteria for through-beam connection detail.

In developing the behavioral model the following assumptions were made:

a. Externally applied shear forces and moments at the joints are known.
b. Failure is defined as the point at which the beam web within the joint reaches its shear stress limit when externally applied forces are at their ultimate values.
c. At failure the concrete stress distribution is linear and maximum concrete compressive stress is below its limiting value.

The joint forces implied in assumption (a.) could be obtained from analysis and requires the knowledge of applied shear and moment at the joint at failure. These quantities are assumed to be related as follows:

$$V_c = \alpha V_b$$
$$M_b = l_1 V_b$$
$$M_c = l_2 V_c$$

where V_b and M_b are ultimate beam shear and moment, respectively, while V_c and M_c are ultimate column shear and moment, respectively. Figure 9 shows these forces for an isolated portion of a structure subjected to lateral loads.

The validity of assumption (c.) above could be justified for the following reasons:

1. Column sizes for the type of construction considered in this paper are generally much larger than the beam sizes.
2. The concrete type used in these columns is generally high-strength concrete with compressive strength well above 10,000 psi. The uniaxial stress-strain characteristics of high-strength concrete exhibit a linear behavior up to maximum strength, followed by a sharp descending portion.

Derivation of Behavioral Model

The type of joint is shown in Figure 9. Figure 10 shows the Free Body Diagram (FBD) of the beam web within the joint

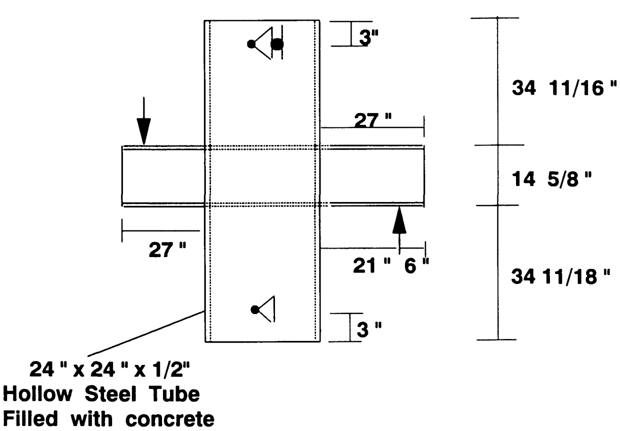

Fig. 5. General configuration of test specimen.

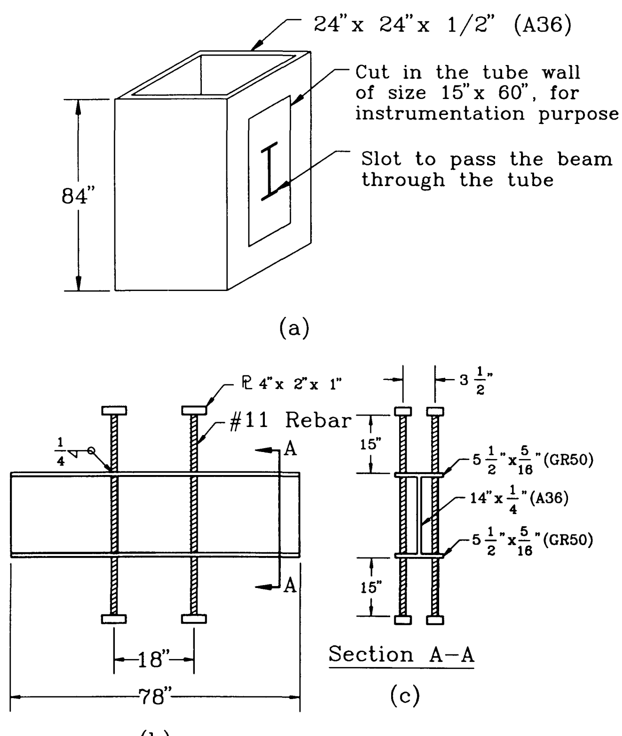

Fig. 6. Different components of the test specimen: a) hollow steel tube, b) hybrid built-up beam section, c) four #11 reinforcing bars with anchor plates welded to each end.

and upper column at ultimate load. With reference to Figure 10, the following additional assumptions are made in deriving the Behavioral Model:

1. The concrete stress distribution is assumed to be linear. The width of the concrete stress block is assumed to equal b_f, beam flange width.
2. As shown in Figure 10, strain distribution over the upper column is assumed to be linear.
3. The steel tube and concrete act compositely.
4. The portion of the upper column shear, V_c, transferred to the steel beam is assumed to be βC_c, where C_c is the resultant concrete compressive force bearing against the beam flange and β is the coefficient of friction.
5. Applied beam moments are resolved into couples concentrated at beam flanges.
6. Resultant of concrete compression strut is along a diagonal as shown in Figure 10.

Considering the above assumptions and strain distribution shown for the upper column in Figure 10, strain for different connection elements could be related to ε_1, steel tube strain in tension.

$$\varepsilon_c = \frac{a}{d_c - a}\varepsilon_1 \tag{1}$$

$$\varepsilon_{sc} = \frac{a - d_1}{d_c - a}\varepsilon_1 \tag{2}$$

$$\varepsilon_{st} = \frac{d_c - d_1 - a}{d_c - a}\varepsilon_1 \tag{3}$$

where

ε_c = maximum compressive strain in steel tube and concrete in compression
ε_{sc} = strain in steel rod in compression
ε_{st} = strain in steel rod in tension

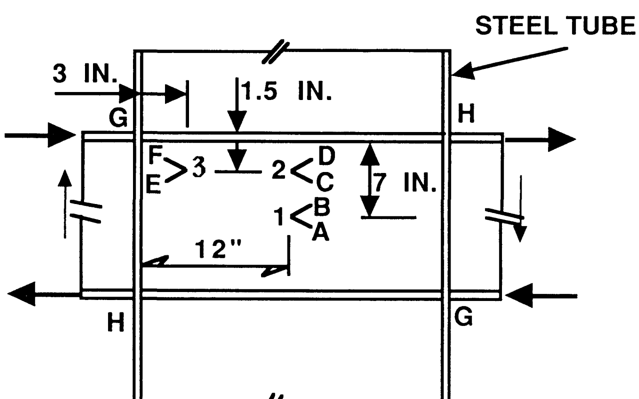

Fig. 7. Location and orientation of gages attached to beam web within the column.

Next, maximum stress in concrete and stresses in the steel rod and steel tube could be calculated as follows:

$$f_c = E_c\varepsilon_c \tag{4}$$

$$f_{sc} = E_s\varepsilon_{sc} \tag{5}$$

$$f_{lc} = E_s\varepsilon_c \tag{6}$$

$$f_{st} = E_s\varepsilon_{st} \tag{7}$$

$$f_{lt} = E_s\varepsilon_1 \tag{8}$$

where f_c, f_{sc}, f_{lc}, f_{st}, and f_{lt} are maximum concrete concrete compressive stress, stress in rod in compression, stress in steel tube in compression, stress in rod in tension, and stress in steel tube in tension, respectively.

Substituting Equations 1 through 3 in Equations 4 through 8 and multiplying Equations 4 through 8 by corresponding area, the resultant forces for different connection elements could be calculated as follows:

$$C_c = \tfrac{1}{2}\eta'\xi b_f \frac{a^2}{d_c - a} f_{y1} \tag{9}$$

$$C_1 = \gamma\xi b_f t_1 \frac{a}{d_c - a} f_{y1} \tag{10}$$

$$C_s = A_s\xi \frac{a - d_1}{d_c - a} f_{y1} \tag{11}$$

$$T_s = A_s\xi \frac{d_c - d_1 - a}{d_c - a} f_{y1} \tag{12}$$

$$T_1 = \xi\gamma b_f t_1 f_{y1} \tag{13}$$

Using the FBD of the upper column shown in Figure 10,

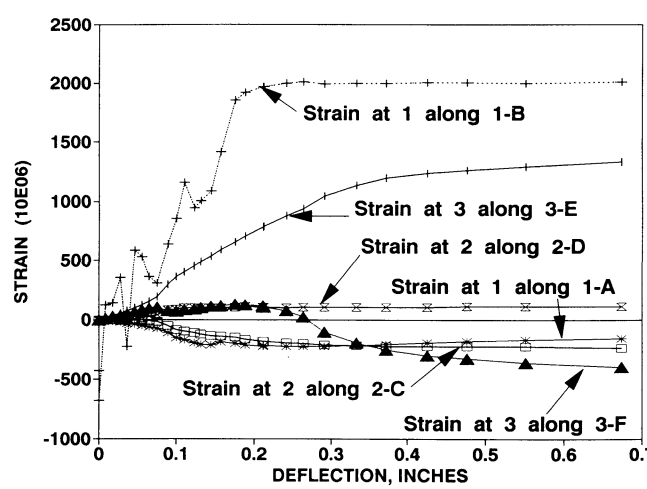

Fig. 8. Strain data from gages attached to beam web within the column.

Equations 9 through 13, and satisfying vertical force equilibrium, the following equation could be obtained.

$$A_s = \frac{1}{d_c - 2a}\left[\tfrac{1}{2}\eta' b_f a^2 - A_1(d_c - 2a)\right] \qquad (14)$$

where

b_f = beam flange width
d_c = depth of the column
a = depth of the concrete compression block
η' = ratio of modulus of elasticity for concrete over modulus of elasticity of steel
A_1 = effective area of steel tube = $2b_f t_1$
A_s = area of steel rod at each corner of the beam
t_1 = thickness of steel tube

In defining A_1 it is assumed that a steel tube width equal to two times the beam flange width is effective in carrying tension and compression. This value was estimated from experimental results.

Next, considering the moment equilibrium of the FBD of the upper column shown in Figure 10 the following expression can be derived.

$$\left[A_1 a d_c + A_s(a d_c - 2d_1 d_c + 2d_1^2) + \tfrac{1}{2}\eta' b_f a^2\left(d_c - \frac{a}{3}\right)\right]\frac{\xi f_{y1}}{\alpha l_2(d_c - a)} = V_b \qquad (15)$$

where

d_1 = distance between steel rod and steel tube
f_{y1} = yield strength of steel tube

In Equation 15 ξf_{y1} is the stress level the steel tube is allowed to approach at ultimate condition. ξf_{y1} could also be viewed as the portion of the steel tube strength utilized to resist the forces transferred by the connection. Based on the limited experimental data obtained from this investigation it is suggested that a value of 0.35 be used for ξ.

Equations 14 and 15 relate the externally applied force, V_b, directly and the externally applied forces V_c and M_c indirectly (through the coefficients α and l_2) to different connection parameters such as A_s, A_1, and a.

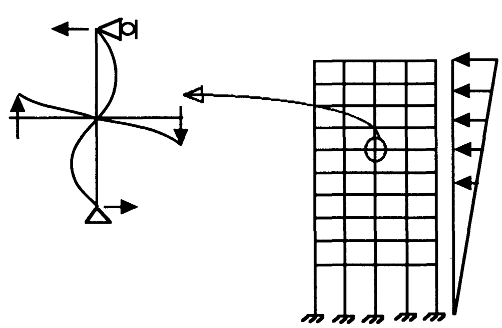

Fig. 9. Assumed forces on an interior joint in a frame subjected to lateral loads.

DESIGN APPROACH

Before proceeding with the steps necessary in designing the through-beam connection detail, additional equations will be derived to relate the shear stress in the beam web within the joint to the compressive force in the concrete compression strut and externally applied forces.

Considering the FBD of a portion of the beam web within

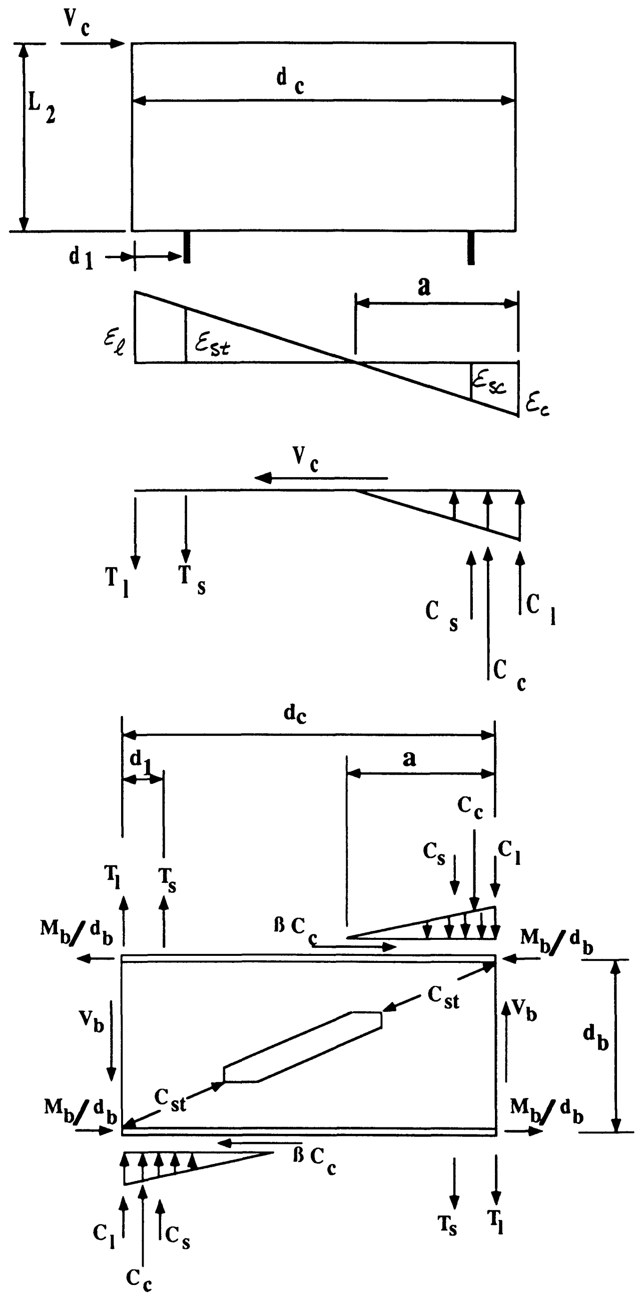

Fig. 10. FBD of the upper column and beam web within the joint area.

the joint area as shown in Figure 11 and satisfying the horizontal force equilibrium, the following equation could be derived:

$$V_w + C_{st}\cos\theta + \beta C_c - \frac{2M_b}{d_b} = 0 \qquad (16)$$

where

V_w = shear force in the beam web at ultimate condition

$\theta = \arctan \dfrac{d_b}{d_c}$

Equations 14, 15, and 16 could be used to proportion the through-beam connection detail.

Until further research is conducted the following steps are suggested for designing the through-beam connection detail following the LRFD format.

Step 1. From analysis, obtain factored joint forces.

Step 2. Select the following quantities: t_1, b_f, d_b, d_c, d_1, f_{y1}

Step 3. Solving Equations 14 and 15 simultaneously, obtain A_s and a. This could be achieved using the trial and error approach.

Step 4. Check stress in different connection elements.

Step 5. Assume the beam web yields at ultimate load. With this assumption V_w could be calculated as follows:

$$V_w = 0.6 F_{yw}\, t_w\, d_c \qquad (17)$$

where

F_{yw} = beam web yield stress
t_w = thickness of the beam web

Step 6. Using Equation 16 calculate C_{st}, compressive force in the concrete compressive strut, and applied shear force to concrete in the joint area.

Step 7. Check shear stress in concrete in the joint area. The limiting shear force could be assumed to be as suggested by ACI 352 [2]:

$$V_u = \phi R \sqrt{f_c'}\, A_e \qquad (18)$$

where

ϕ = 0.85
R = 20, 15, and 12 for interior, exterior, and corner joints, respectively
f_c' = concrete compressive strength

It is suggested that the value of $\sqrt{f_c'}$ be limited to 100 psi, implying that in the case of 15,000 psi concrete, for instance, $\sqrt{f_c'}$ be taken as 100 rather than 122 as would be obtained from V_u calculations.

Until further research is conducted it is suggested that A_e be calculated as follows:

$A_e = 2b_f \times d_c$

DESIGN EXAMPLE

Design a through-beam connection detail with the following geometry and properties.

Given (Steps 1 and 2):

t_1 = 0.5 in.
b_f = 5.5 in.
d_b = 14.5 in.
d_c = 24 in.
d_l = 3.5in.
f_{y1} = 36 ksi
F_{yw} = 36 ksi
t_w = 0.25 in.
α = 0.85
l_2 = 32 in.
V_b = 79 kips
M_b = 1,660 in-kips
β = 0.5
ξ = 0.35
η' = 0.23
A_1 = 5.5 in ^{2}
f_c' = 14 ksi
E_s = 29,000 ksi (modulus of elasticity of steel)
E_c = 6,670 ksi (modulus of elasticity of concrete)

Step 3: Using the trial and error approach and Equations 14

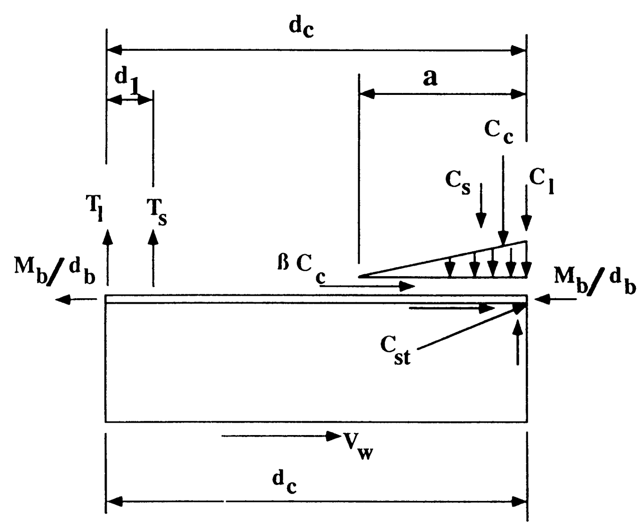

Fig. 11. FBD of the portion of the web within the joint area.

and 15, calculate a and A_s. For the first trial assume $a = 8.5$. Equation 14 will result in:

$$A_s = \frac{1}{24 - 2 \times 8.5} [½(0.23)(5.5)(8.5)^2 - 5.5(24 - 2 \times 8.5)$$

$$A_s = 1.03 \text{ in}^2$$

Substitute $A_s = 1.03$ in.^{2} in Equation 15 and calculate V_b. If the result is approximately equal to 79 kips the assumed value of a is **o.k.** Equation 15 yields:

$$V_B = [5.5 \times 8.5 \times 24 + 1.03(8.5 \times 24 - 2 \times 3.5 \times 24 + 2 \times 3.5^2) + ½(0.23)(5.5)(8.5)^2(24 - 8.5/3)] \frac{0.35 \times 36}{0.85 \times 32(24 - 8.5)}$$

$$V_b = 64.3 \text{ kips} \neq 79 \text{ kips}$$

Assume a = 9 inches. This will yield A_s = 3.04 in.^{2}, $V_b = 77^k \approx 79^k$ **o.k.**

Therefore, a = 9 inches and A_s = 3.04 in.^{2}

Use two #11 Grade 60 deformed reinforcing bars. A_s = 3.12 in.^{2}

Step 4: Check stresses in different connection elements against their limit values. First calculate tensile strain in the steel tube.

$$\varepsilon_1 = \xi f_{y1} / E_s = 0.35 \times 36/29{,}000 = 0.000434 \text{ in./in.}$$

Using Equations 1 and 4 calculate f_c.

$f_c = 1.74 \text{ ksi} < f_c' = 14 \text{ ksi}$ **o.k.**

Using Equations 2, 3, 5, 6, 7, and 8 calculate stresses in other connection elements. This yields:

$f_{sc} = 4.61 \text{ ksi} < \phi_c F_y = 0.85 \times 60 = 51 \text{ ksi}$ **o.k.**
$f_{1c} = 7.55 \text{ ksi} < \phi_c F_y = 0.85 \times 36 = 30.6 \text{ ksi}$ **o.k.**
$f_{st} = 9.65 \text{ ksi} < \phi_t F_y = 0.9 \times 60 = 54 \text{ ksi}$ **o.k.**
$f_{lt} = 12.6 \text{ ksi} < \phi_t F_y = 0.9 \times 36 = 32.4 \text{ ksi}$ **o.k.**

Step 5: Using Equation 17 calculate shear force in the beam web:

$$V_w = 0.6 \times 36 \times 0.25 \times 24 = 129.6 \text{ kips}$$

Step 6: Using Equation 16 calculate compressive force in concrete compression strut.

$\theta = \arctan 14.5/24 = 31.1°$
$C_c = ½\eta'\xi b_f (a^2 / d_c - a) f_{y1}$
$C_c = ½(0.23)(0.35)(5.5)(9^2 / 24 - 9) \times 36 = 43 \text{ kips}$
$V_w + C_{st}\cos(\theta) + \beta C_c - (2M_b / d_b) = 0$
$129.6 + C_{st}\cos(31.1) + 0.5(43) - (2 \times 1{,}660) / 14.5 = 0$
$C_{st} = 90.9 \text{ kips}$

Step 7: The shear force carried by concrete within the joint between the beam flanges is assumed to be the horizontal component, C_{st}.

$V_c = C_{st}\cos(\theta)$
$V_c = 90.9 \cos(31.1) = 77.8^k$

For the interior joint the shear capacity is

$V_u = \phi(20) f_c'(2b_f)(d_c)$
$V_u = 0.85(20)100\times[(2\times5.5)(24)]/1{,}000 = 449^k > 77.8^k$ **o.k.**

SUMMARY AND CONCLUSIONS

The use of composite columns of the type described in this paper is proven to be economical. This paper has summarized a suggested connection detail (a through-beam connection detail) for connecting steel beams to these columns as well as tentative design guidelines. The information presented in this paper is based on a pilot study and, therefore, it is suggested that this information be viewed as a general guideline until further research is carried out. It should also be noted that the effect of axial load in the column on performance of the connection was not considered. The intent of the paper is to suggest an economical connection detail and outline a procedure to comprehend its behavior through the behavioral model presented.

ACKNOWLEDGMENT

The authors greatly appreciate the support provided by the American Institute of Steel Construction which provided partial funding of a graduate student. Valmont Industries of Omaha, Nebraska is greatly appreciated for constructing the test specimen. Special thanks are also due the Center for Infrastructure Research at the University of Nebraska-Lincoln for supporting this research. The authors thank Dr. J. P. Colaco of CBM Engineers, Inc., Houston, Texas for his helpful suggestions and great encouragement while pursuing this research.

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