Composite Column Force Transfer in Special Two-Story X-Braced Frames

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Abstract: Reseach results have shown that columns in low-to-medium rise special two-story X-braced frames should be designed based on the capacity of the braces because of high column demands. Composite columns, either encased of filled, can be an economical solution to deal with the additional axial load capacity over that available with steel columns alone. Forces must be transferred between steel and concrete in composite columns so that dissimilar materials can achieve a state of equilibrium and act in a composite manner. With the new 2010 AISC Specification for Structural Steel Buildings, the provisions for force transfer have been greatly expanded in a new section to address the lack of clarity in previous Specification. The concept of load introduction length (AISC Specification I6.4) will be applied in the design the composite columns in low-to-medium rise special two-story X-braced frames with particular emphasis on the headed stud anchor provisions as they relate to load transfer. The clearer guidance in addressing component behaviour, design techniques, and proper detailing considerations in the 2010 AISC Specification will be followed.

Keywords: 2010 AISC Specification, column demand, composite column, special concentrically braced frame, two-story X-configuration braces.

1 INTRODUCTION

1.1 Seismic Column Demands in Special Two-Story X Braced Frames

Special concentrically braced frames (SCBF) have been known as a very efficient and economical system for resisting lateral forces and minimizing building drifts. SCBFs are efficient because framing members resist primarily axial loads with little or no bending in the members until the compression braces in the system buckle. Brace buckling is allowed because special gusset plate detailing is required for both in-plane and out-of-plane brace buckling design, depending on brace buckling mode selected.

The two-story X-braced frame could be a better alternative to the V or inverted V braced frame because the two-story X configuration braces prevents the development of unbalanced forces on the beam, and distributes this unbalanced vertical load to other levels that are not experiencing high seismic demands providing for better overall frame performance (Figure 1).

A research done by Richards(2009) showed that in low-rise SCBFs with braces in the two-story X-configuration column axial demands were up to 100% greater than those commonly used in the design because of force redistribution that occurs after brace buckling. The results of this research showed that the two-story X-braced configuration is not necessarily a better or safer alternative to the V or inverted V configuration because of the high seismic axial demands.
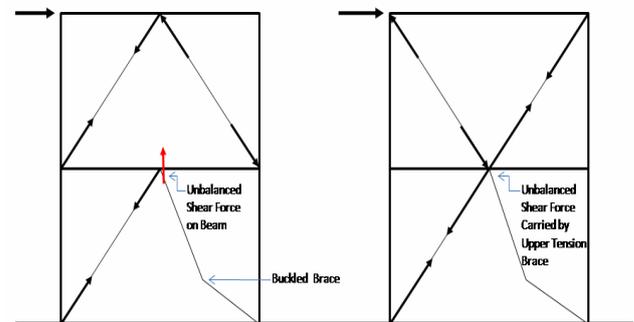

Figure 1. Brace configuration and unbalanced shear force: inverted V and two-story X configuration braces.

1.2 Concrete Encased Composite Column and Force Transfer

Due to the high column demands in the two-story X-braced frames, very heavy columns would be the results of the design if bare steel columns are employed. Therefore composite columns, either encased of filled, can be an economical solution to deal with the additional axial load capacity over that available with steel columns alone.

Force transfer deals with the balancing forces at the interface of the steel and concrete in a composite column so the steel and concrete can achieve an internal equilibrium and act in a composite manner. The 2010 AISC Specification is now provide a clearer guidance for the allocation of forces between steel and concrete as well as the force transfer mechanisms used for composite columns. The guidance in addressing component behaviour, design techniques, and proper detailing considerations will be followed in this paper.

2 AXIALLY LOADED CONCRETE ENCASED COMPOSITE COLUMNS

Composite columns can typically be categorized as filled composite columns, known as concrete filled tubes (CFTs), and encased composite columns, also known as steel reinforced concrete (SRC) columns. Although the behaviours of SRC columns and CFT columns are based on the same general principles, there are enough differences, especially with regard to details. This paper discusses the application of SRC columns for special two-story X-braced frames.

2.1 Strength of SRC Columns

The section strength P_{no} of SRC columns can be taken as the summation of the axial compressive strenghs of the component materials as follows:

$$P_{no} = A_sF_y + A_{sr}F_{ysr} + 0.85A_cf_c' \quad \text{(2010 AISC I2-4)}$$

where:
A_s = area of th steel section, mm^2
A_c = area of encasement, mm^2
A_{sr} = area of continuous reinforcing bars, mm^2
f_c' = specified compressive strengh of concrete, MPa
F_y = specified minimum yield stress of steel, MPa
F_{ysr} = specified minimum yield reinforcing bars, MPa

A plastic stress distribution model is used to determine the strength of SRC columns where both the steel section and the reinforced concrete section are assumed to reach the ultimate strengh concurrently. Because of slenderness effects, the strength predicted by 2010 AISC Equation I2-4 cannot be achieved. To account for slenderness, the relationship between P_{no} and P_e is used, where P_e is the Euler buckling load and is defined as

$$P_e = \frac{\pi^2(EI)_{eff}}{(KL)^2} \quad \text{(2010 AISC I2-5)}$$

where $(EI)_{eff}$ is the effective flexural rigidity of the composite section and is given by

$$(EI)_{eff} = E_sI_s + 0.5E_sI_{sr} + C_1E_cI_c \quad \text{(2010 AISC I2-6)}$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \quad \text{(2010 AISC I2-7)}$$

The nominal strength P_n is calculated as follows:

When $P_e \geq 0.44P_{no}$

$$P_n = P_{no}\left[0.658^{\left(\frac{P_{no}}{P_e}\right)}\right] \quad \text{(2010 AISC I2-2)}$$

Otherwise for $P_e < 0.44P_{no}$
$$P_n = 0.877P_e \quad \text{(2010 AISC I2-3)}$$

For LRFD, the design strength is ϕ_cP_n where $\phi_c = 0.75$

2.2 Force Introduction to SRC Columns

A load path is needed for introduction of external forces to SRC columns. Shown in Figure 2 below is the the concept of force introduction to SRC columns illustratd by William and Hajjar(2010). The external forces are typically applied to the SRC columns through direct connection to the steel member (Figure 2a), bearing on the concrete (Figure 2b), or a combination thereof (Figure 2c).

(a)
Load Transfer to SRC with Steel Connection

(c)
Load Transfer to SRC via Concrete and Steel Concurrently

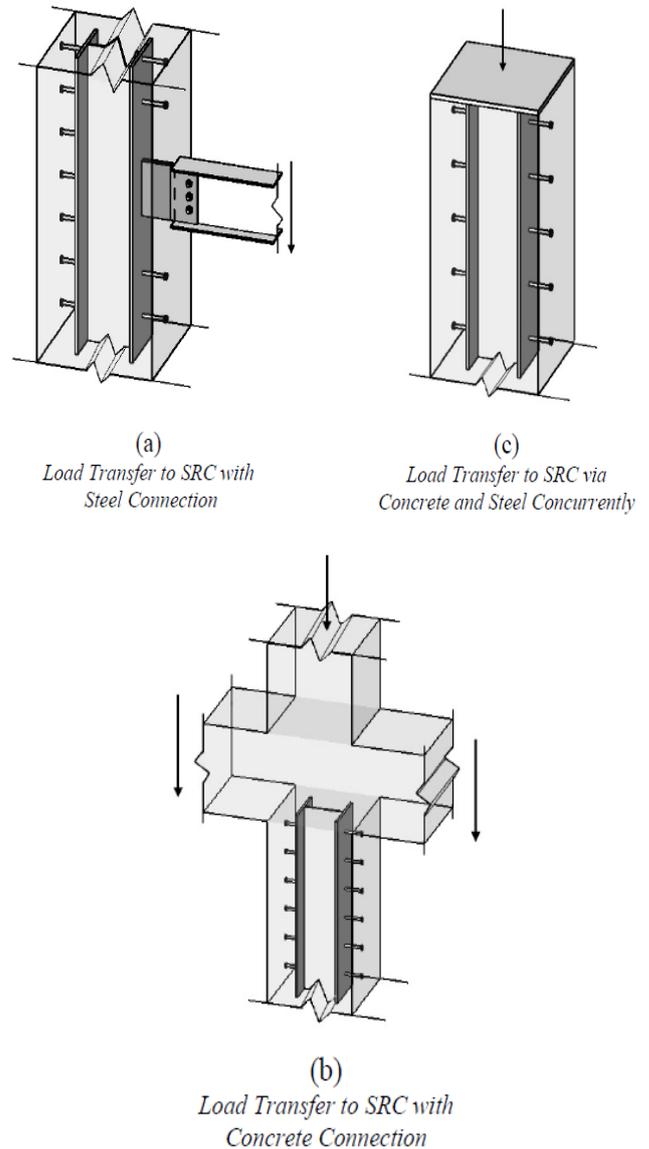(b)
Load Transfer to SRC with Concrete Connection

Figure 2. Examples of Force Introduction to SRC Columns(William and Hajjar(2010))

Once the load path has been provided, the interface between the steel and concrete should be designed to transfer the longitudinal shear required to maintain the internal equilibrium within the SRC column section. The equations form of the 2010 AISC Specification for load transfer to SRC columns, that can be used to calculate the required longitudinal shear, are as follows:

For entire external force applied directly to the steel section (Figure 2a):

$$V_r' = P_r \left(1 - \frac{As\,Fy}{Pno} \right) \qquad \text{(2010 AISC I6-1)}$$

For entire external force applied directly to the concrete section (Figure 2b):

$$V_r' = P_r \left(\frac{As\,Fy}{Pno} \right) \qquad \text{(2010 AISC I6-2)}$$

where

V_r' = required longitudinal shear force to be transferred, N
P_r = required external force applied to the composite member, N
A_sF_y = plastic capacity of steel section, N
P_{no} = plastic capacity of composite section determined by 2010 AISC equation (I2-4), N

When external force is applied to both materials concurently, 2010 AISC Specification provides two options. The longitudinal shear force to be transferred to achieve cross-sectional equilibrium can be taken as either the difference in magnitudes between the portion of external force applied directly to the concrete and that required by equation I6-1 or the portion of external force applied directly to the steel section and that required by equation I6-2. William and Hajjar(2010) expressed the second option as follows:

$$V_r' = P_{rs} - P_r \left(\frac{As\,Fy}{Pno} \right) \qquad (1)$$

where

P_{rs} = portion of external force applied directly to the steel, N

One possible method of determining P_{rs} in a bearing plate condition (Figure 2c) recommended by William and Hajjar(2010) is to assume the force is initially applied to each material in relation to their axial stiffness (the ratio of the area times the modulus elasticity for each material).

2.3 Force Transfer in SRC Columns

The 2010 AISC Specification Section I6.3 provides a clearer mechanism for force transfer in composite columns. For concrete encased composite columns, the longitudinal shear force to be transferred (V_r') is permitted via direct bearing or shear connection. Transfer of longitudinal shear via direct bond interaction is permitted solely for CFT columns.

2.3.1 Direct Bearing

Direct bearing refers to the use of bearing plate or other similar assemblies (probably via beam ends at

beam-column joints) to transfer the required longitudinal shear force in composite colums as illustrated in Figure 3 for a CFT column with an internal bearing ring.
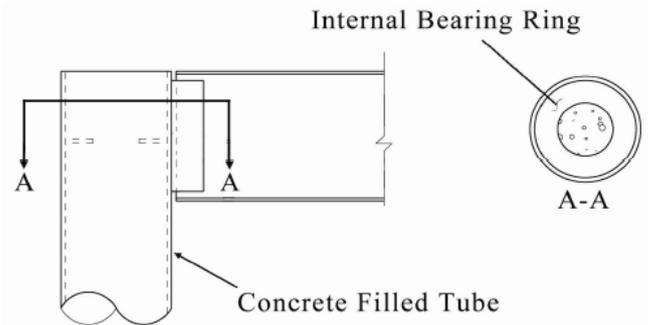

Figure 3. CFT with Internal Bearing Ring(William and Hajjar(2010))

2010 AISC Specification I6.3a stipulates the available bearing strength of the concrete be determined as follows:

$$R_n = 1.7\,f_c'\,A_1 \qquad \text{(2010 AISC I6-3)}$$
$$\emptyset = 0.65$$

Where A_1 = loaded area of concrete, mm2

2.3.2 Shear Connection

2010 AISC Specification Section I8.3 cover the requirement for steel anchors in composite components. This section apply to the design of cast-in-place steel headed stud anchors and steel channel anchors in composite components. Headed studs are normally used as a force transfer mechanism in concrete encased composite columns. For steel headed stud anchors in normal weight concrete subjected to shear only, the AISC Specification limits the length from the base of the headed stud to the top of the stud head after installation not less than five stud diameter. Shear strength of headed stud anchor in a composite column is determined as follows:

$$R_n = A_{sc}F_u \qquad \text{(2010 AISC I8-3)}$$
$$\emptyset = 0.65$$

where

R_n = nominal shear strength of single headed stud anchor
A_{sc} = cross-sectional area of the steel headed stud anchor, mm^2
F_u = Specified minimum tensile strength of stud anchor, ksi (MPa)

The needed quantity of headed stud anchors for force transfer is determined by:

$$N_{anchors} = \frac{Vr'}{\emptyset Rn}$$

where

$N_{anchors}$ = required number of steel headed stud anchors for force transfer

For conrete encased composite columns, the steel headed stud anchors shall be placed on at least two faces of the steel shape in a symmetric configuration about the steel shape axis.

2.3.3 Load Introduction Length

To avoid overstressing the structural steel section of the concrete at connections in encased composite column, 2010 AISC Specification requires the longitudinal shear transfer to occur within the load introduction length shown at Figure 4. It is important for the shear transfer to take place as quickly as possible to facilitate composite action (William and Hajjar(2010) so the 2010 AISC specification limits the load introduction length to two times the minimum transverse dimension both above and below the load transfer region. The load transfer region is not explicitly defined within the AISC Specification, but may be interpreted to equal the depth of the connection introducing the external force. The headed stud anchors required for longitudinal shear transfer are located within the load introduction length.
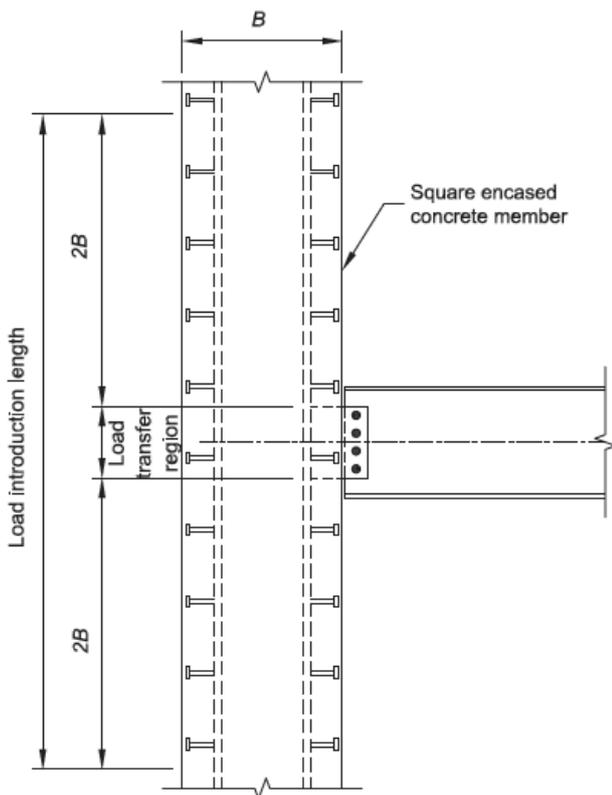

Figure 4. Load Introduction Length(AISC 360-10)

2.4 Strength and Ductility of Concrete Encased Composite Columns

El-Tawil and Deierlein(1999) studied the strength, stiffness and ductility of encased composite columns using fiber section analysis shown in Figure 5.
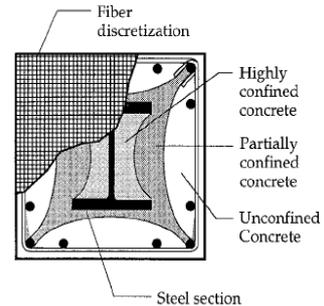

Figure 5. Fiber Idealization of Concrete Encased Composite Column (El-Tawil and Deierlein(1999))

Three sections shown on Figure 6 are used as prototypical design examples to investigate the strength and stiffness of encased composite columns cross sections. Reinforcing bars and structural steel sections have yield strength of F_{yr} = 414 Mpa and F_{ys} = 345 MPa respectively. Three concrete strengh are used - f_c' = 28, 69, and 110 MPa representing low-, medium-, and high-strength concrete. Different encased shapes with structural steel ratio of $\frac{A_s}{A_g}$ = 0.04, 0.08, and 0.16 were studied. . The naming convention reflects the steel ratio and concrete strength (e.g., S-08-M refers to a section with a steel ratio of $\frac{A_s}{A_g}$ = 0.08 and medium-strength concrete).
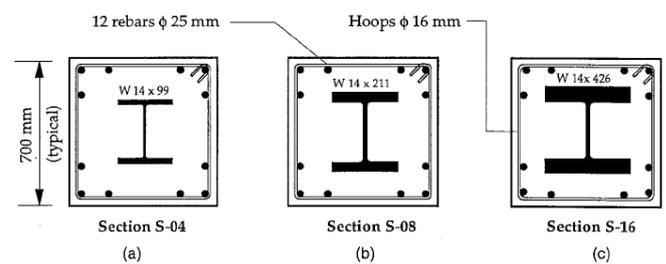

Figure 6. Prototype Composite Columns (El-Tawil and Deierlein(1999)): S-04; (b) S-08; (c) S-16

The seismic hoop reinforcement shown in Figure 7, is investigated by El-Tawil and Deierlein(1999) to evaluate confinement effects on the strength and ductility of composite columns. This reinforcement consists of 16-mm diameter hoops with four branches, spaced along the column at 100 mm on center for concrete with f_c' = 28 and 69 MPa and at 75 mm on center for f_c' = 110 MPa concrete.
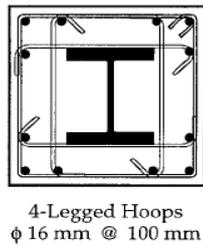
4-Legged Hoops
ϕ 16 mm @ 100 mm

Figure 7. Details of Seismic Hoop Reinforcement for S-08 (El-Tawil and Deierlein(1999))

The conclusion of the evaluation done by El-Tawil and Deierlein(1999) for the concrete encased composite columns were as follows:

1. Composite columns with normal strength concrete ($f_c^{'}$ = 28 MPa) had curvature ductilities on the order of $\mu_\phi = 4 - 12$ when subjected to intermediate to high axial load levels ($P = 0.3 - 0.6P_o$).

2. Ductility improved significantly when confinement steel was provided by the transverse hoop reinforcement specified in the AISC Seismic Provisions for composite columns (Figure 7).

3. The compression load $P = 0.6P_{no}$ is about the maximum that should ever occur in a design.

4. The presense of a large steel core provides a beneficial residual strength following concrete crushing and leads to improve ductility. Columns with encased shapes benefit from the confinement of the concrete between the column flange (Figure 5).

3 A WORKED EXAMPLE OF CONCRETE ENCASED COMPOSITE COLUMN FORCE TRANSFER IN SPECIAL TWO-STORY X-BRACED FRAMES

3.1 Composite Special Concentrically Two-Story X-Braced Frames

Care should be taken when computing maximum possible column demand in frames with two-story X-bracing. Richards(2009) showed that the column demands for these frames when brace is removed, which is analogous to buckling, double even with the same floor forces (Figure 8).

A possible approach to design column in frames with two-story X-bracing is to design to the column based on maximum load the can be delivered by the braces. Bracing member sections are determined from the stiffness and strength requirement of the lateral system. Rectangular HSS braces, which are aesthetic as architecturally exposed elements, are used in this worked example.
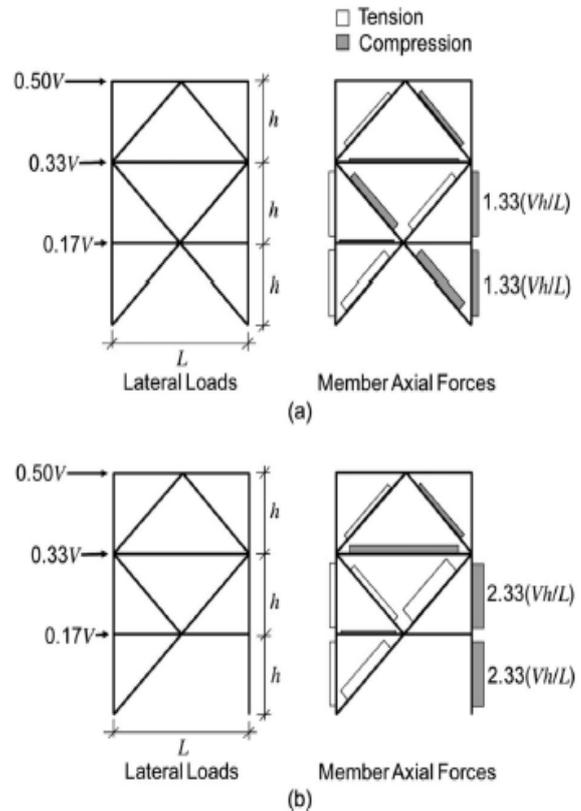

Figure 8. Forces in SCBF with two story X-bracing (Richards(2009)): (a) before brace removal; (b) after brace removal

The strength and ductility requirement in concrete encased composite columns can be achieved by fullfilling the four points recommended by El-Tawil and Deierlein(1999) stated previously. In SCBFs with braces in the two story X-configuration where heavy column loads are being supported, concrete can be added to carry additional load without requiring an increase in the size of the steel section. However heavier steel section might be required so that the limit of compression load $P \leq 0.6P_{no}$ is fullfilled.

The following load combinations are used to calculate the column demands:

1. Strength design:
   1.2xD + 0.5xL $\pm$ 1.0xE$_h$

2. Ductility design:
   1.2xD + 0.5xL $\pm$ (bracing capacities)

3.2 A Worked Example

Show in Figure 9 below is the elevation view of the lateral resisting system of a five-story office building constructed at a hard soil in zone 6 region of Indonesian seismic map. All braced bays have the two-story-X configuration. W shapes (F_y = 350 MPa) are used for all beams and columns.
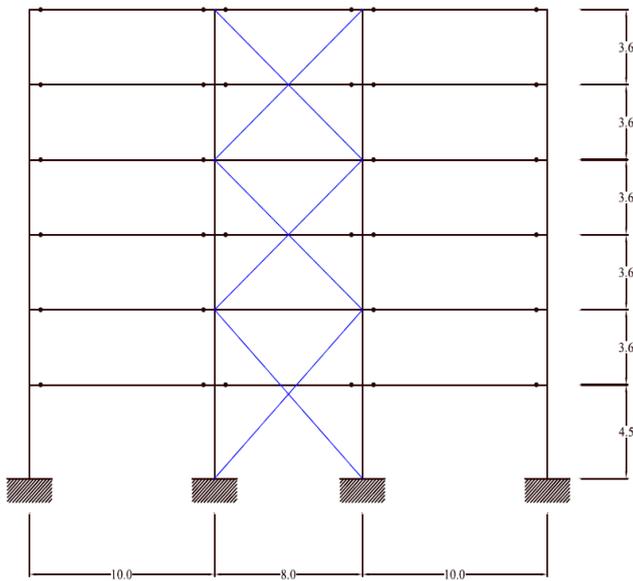
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Figure 9. Elevation View

Square HSS (F_y = 46 Ksi) braces are selected based on $\frac{K.L}{r} \leq 4\sqrt{\frac{E}{Fy}}$ and $b/t < 6.4\sqrt{\frac{E}{Fy}}$. The compressive and tensile capacities of the braces, corresponding to each floor of the building based on 2005 AISC 341 Section 13.3b and section 13.3c, are shown in Table 1.

Table 1. Bracing Capacities (2005 AISC 341)

FL	Square HSS	$R_yF_yA_g$ (kN) Section 13.2b	$1.1R_yP_n$ (kN) Section 13.3c
6	HS178x178x10	+2744.41	-1983.19
5	HS178x178x11	+3152.97	-2296.33
4	HS203x203x13	+4112.18	-3264.75
3	HS203x203x13	+4112.18	-3264.75
2	HS203x203x13	+4112.18	-3264.75
1	HS203x203x13	+4112.18	-2879.12

The maximum column demands can be obtained using joint equilibrium, starting from top to bottom, based on the bracing capacities. To utilize the composite behaviour of the column, vertical component of brace force must be transferred through the beam-brace-column connection and distributed to both concrete and steel in the composite column as shown in Figure 10 below. By calculating the maximum column demands based on the maximum loads the can be delivered by the braces meaning the composite columns are capacity designed to the strength of the braces.
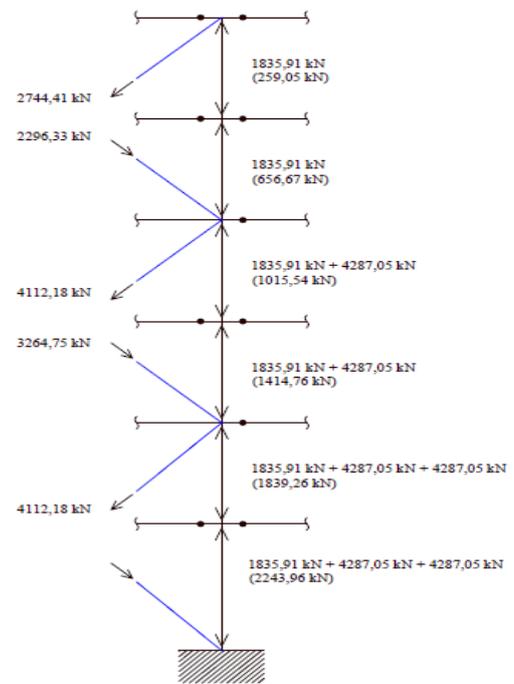

Figure 10. Maximum Column Demands

In columns of braced frames, when the columns are capacity designed to the strength of the diagonals, axial forces are more significant than bending moment. The axial force in composite columns must be shared between the steel section and the encased concrete in particular zone where the column axial force is the highest (at the floor levels where bracings and beams are connected to the columns).

The AISC 2010 equations I2-2 to I2-7 are combined in a spreadsheet and used to calculate the strength of the encased concrete composite columns. The results of the strength and ductility design can be seen in Table 2. Due to high axial demand, very heavy columns would be the result of the ductility design with steel columns alone. The required headed stud anchors for longitudinal shear transfer in the composite columns at the second story are determined following the design procedures in the new 2010 AISC Specification. The calculations at other stories follow the same procedures.

The encased shape of the composite column at the second story has the following components:

Steel core: W360x162
  A_s = 20600 mm^2
  E_s = 200.000 MPa
  F_y = 350 MPa

Reinforcing bars: 12D25
  A_{yr} = 5880 mm^2
  E_s = 200.000 MPa
  F_{yr} = 414 MPa

Concrete: 750x750 mm^2

$E_c = 4700\sqrt{f_c'} = 24870$ MPa

$A_c = 750$x$750 - 5880 - 20600 = 536020$ mm^2

The section strength of the composite column, P_{no}, is calculated using 2010 AISC equation I2-4:

$P_{no} = A_sF_y + A_{sr}F_{ysr} + 0.85A_cf_c'$

$= 20600$x$350 + 5880$x$414 + 0.85$x536020x28

$= 22401596$ N $= 22401.6$ kN

$P_r = 12897.25$ kN (Table 2) is the external force applied to the composite member.

There are two options to calculate V_r', the required longitudinal shear force to be transferred:

Option 1: Assume the external force (P_r) is applied to the steel section (Figure 2a). Calculate P_r using 2010 AISC equation I6-1. This assumption is conservative .

$V_r' = P_r(1 - \frac{A_s F_y}{P_{no}}) = 12897.25 (1 - \frac{20600x350}{22401596})$

$= 8746.24$ kN

Option 2: Assume the external force (P_r) is applied to both materials concurently (Figure 2c).

$V_r' = P_{rs} - P_r(\frac{A_s F_y}{P_{no}})$

William and Hajjar(2010) recommends P_r is initially applied to each material proportional to their axial stiffness.

P_{rs} = portion of P_r applied directly to the steel

$= \frac{E_s A_s}{(E_s A_s + E_c A_c)}$ x P_r

$= \frac{200000 \ x \ 20600}{(200000 \ x 20600 + 24870 \ x \ 536020)}$ x 12895.25

$= 0.236$ x $12895.25 = 3043.22$ kN

$V_r' = 3043.22 - 12897.25x(\frac{20600 \ x \ 350}{22401596})$

$= -1107.8$ kN. The minus sign indicates the longitudinal shear force that must be transferred from portion of the force applied directly to the concrete is bigger.

The different value of V_r' from these two assumptions is too large. Moreover the new 2010 AISC Specification doesn't give a clear cut guidance about determining the P_{rs}. Therefore to be on the safe side, the required longitudinal shear to be transferred between concrete and steel section is taken as $V_r' = 8746.24$ kN.

V_r' will be transferred via shear connection. The superposition of transfer mechnisms is not allowed by 2010 AISC Specification as experimental data indicate that the shear connection often does not initiate until after direct bond interaction has be breached. The direct bond between steel and concrete is ignored. One could argue that the beam ends at the beam-column joint simulate the bearing plate used for longitudinal shear transfer via direct bearing (Figure 2c). However there is no clear cut guidance to deal with this possibility in new 2010 AISC Specification.

William and Hajjar(2010) showed that Palarés and Hajjar(2010) had done a detailed review of the relevant data of headed stud anchors. The headed stud anchors in shear, in a normal weight concrete, with a minimum h/d (total length over shaft diameter) ratio of five failed through steel strength failure in over 80% tests. Steel headed stud anchor limits and shear strength for composite components has been grouped in Table 3.

Try steel anchor 7/8 x 5 3/16:

The shear strength of each stud is R_n=113 kN

$N_{anchors} = \frac{V_r'}{\emptyset R_n} = \frac{8746.24}{0.65 \ x \ 113} = 119$ studs

The 119 headed stud anchors required for longitudinal shear transfer will be distributed within the load introduction length. In this worked example, the load transfer region is about 400 mm. Therefore the load introduction length (Figure 4) is 2x750 + 2x625 + 400 = 3150 mm. Two headed stud anchors will be installed at each flange of the composite column. Therefore the spacing among the stud is:

Studs spacing = $\frac{3150}{119/4} = 105.88$ mm ~ 100 mm

2010 AISC Specification stipulates the minimum center-to-center spacing of the headed stud anchors is 4 diameter = 88 mm, and the maximum stud spacing center-to-center is 32 diameter = 704 mm. Therefore the 120 studs will be spaced 100 mm, two studs at each flange of the composite column.

For concrete encased composite columns, headed stud anchors are required throughout the column length in order to maintain composite action of the column under incidental moment. Therefore outside the load introduction length, headed stud anchors with maximum spacing (700 mm) are provided.

Table 2. Maximum Possible Columns Demands (kN), Bare Steel Columns, and Composite Columns

	Strength Design $1.2\text{x}D + 0.5\text{x}L \pm 1.0\text{x}E_h$		Ductility Design (bare steel columns) $1.2\text{x}D + 0.5\text{x}L \pm$ (bracing cap)		Ductility Design (composite columns) $1.2\text{x}D + 0.5\text{x}L \pm$ (bracing capacities)		
					Dimension	W Shapes	Reinforcing Bars, F_{yr}=414 MPa
6	230.71	W200x46	2094.98	W200x100	400x400mm^2	W200x46	4 D16mm
5	493.45	W200x46	2492.58	W200x100	400x400mm^2	W200x46	4 D16mm
4	1092.82	W250x58	7138.92	W310x226	625x625mm^2	W250x73	4 D25mm
3	1356.95	W250x58	7537.74	W310x226	625x625mm^2	W250x73	4 D25mm
2	2195.66	W310x97	12897.25	W360x382	750x750mm^2	W360x162	12 D25mm
1	2465.26	W310x97	13301.85	W360x382	750x750mm^2	W360x162	12 D25mm

Table 3. Normal Weight Concrete: Steel Headed Studs Anchor Limits and Shear Strength for Composite Components(Pallarés and Hajjar(2010))

Steeel Anchor Diameter in (mm)	Steel Anchor Area in^2 (mm^2)	Minimum Length "h" In (mm)	Corresponding Standard Stock Shear Connector	Available Shear Strength LRFD Kips (kN)
¾ (19)	0.44 (284)	3 ¾ (95)	¾ x 4 3/16	18.7 (83)
7/8 (22)	0.60 (380)	4 3/8 (110)	7/8 x 5 3/16	25.4 (113)
1 (25)	0.79 (491)	5 (125)	1 x 6 ¼	33.2 (148)

4 CONCLUSIONS

The clearer guidance in the expanded provisions for force transfer in the 2010 AISC Specification has been shown operational in the design of concrete encased composite columns in the two-story X-braced frames which have high column demands. However further decisive guidance is still needed for force transfer when external force is applied to both materials concurently.

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