Composite Columns in Low-to-Medium-Rise SCBFs with the Two-Story X-Configuration Braces

Junaedi Utomo[1]

[1]*Civil Engineering Department, Universitas Atma Jaya Yogyakarta*

Abstract

Column demands of special concentrically braced frames (SCBFs) were investigated by Richards (2009). In low-rise SCBFs with braces in the two-story X-configuration, column demands were up to 100% greater than those commonly used in the design because of force redistribution that occurs after brace buckling. The 2005 AISC Seismic Provisions do not require columns to be designed for the maximum force that can be delivered to them if $\frac{kl}{r} \leq 4\sqrt{\frac{E}{Fy}}$. *If* $\frac{kl}{r} \leq 4\sqrt{\frac{E}{Fy}}$ *columns are designed for loads corresponding to twice the axial loads caused by the design base shear (*$P_u = \Omega_o \times P_{base\_shear}$*, where* $\Omega_o = 2$*). This approach is based on engineering judgement that is need to be questioned for the SCBFs with braces in the two-story X-configuration. The design of the columns should be done based on the maximum load that can be delivered by the braces. Composite columns, either encased of filled, can be an economical solution for the very high column demands. The concrete in the composite column can be added to carry additional loads without requiring an increase in the size of the steel section. The 2005 AISC Specification for Structural Steel Buildings provides the simple and practical methods to determine the capacity of composite columns. This specification allows composite columns to be designed with a minimum of 1% steel ratio, down from the 4% required in previous LRFD specifications. Very heavy columns would be the results of the design if bare steel columns instead of composite columns are employed. The design of composite columns using the 2005 AISC Specification will be discussed.*

Key Words: *2005 AISC Specification, column demand, composite column, special concentrically braced frame, two-story X-configuration braces*

Introduction

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.

When the compression brace buckles in a V or inverted V configuration, the beam at the mid-span connection must deflect downward because of the unbalanced forces on the beam. This deflection can result in significant damage to the slab system attached to the beam. It can be implied from the AISC Seismic Design Manual (….) that the two-story X-braced frame is 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.

A research done by Richards(…) 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 additional axial load capacity. 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. The columns in special two-story X-braced frames should be designed based on the capacity of the braces.

Strength and Ductility of Concrete Encased Composite Columns

Composite columns can take one of two forms: a pipe or HSS filled with plain concrete or a rolled steel shape encased in concrete with both vertical and transverse reinforcement. Although the behaviors of encased and filled composite columns are based on the same general principles, there are enough differences, escpecially with regard to details, that the AISC Specification threats them separetely. This paper discusses the application of concrete encased composite columns to special two-story X-braced frames.

Strength of Encased Composite Columns

If buckling were not an issue, the column strength could be taken as the summation of the axial compressive strengths of the component materials:

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f_c' \qquad \text{(AISC Equation I2-4)}$$

Because of slenderness effects, the strength predicted by AISC Equation I2-4 cannot be achieved. To account for slenderness, the relationship between P_o and P_e is used, where P_e is the Euler buckling load and is defined as

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

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

$$(EI)_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \qquad \text{(AISC Equation I2-6)}$$

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

The nominal strength is calculated as follows:

When $P_e \geq 0.44 P_o$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right] \qquad \text{(AISC Equation I2-2)}$$

When $P_e < 0.44 P_o$

$$P_n = 0.877 P_e \qquad \text{(AISC Equation I2-3)}$$

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

Curvature Ductility of Encased Composite Columns

El-Tawil and Deierlein(…) studied the strength, stiffness and ductility of concrete encased composite columns using fiber section analysis (Fig. 1). Three sections shown on Fig. 2 was 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). The transverse reinforcement in Fig. 2, consisting of 16-mm diameter ties spaced at 320 mm, meets the standard (nonseismic) ACI 318 requirements.

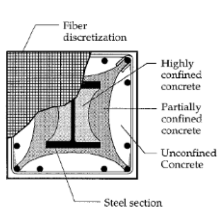

Fig.1. Fiber Idealization of Concrete Encased Composite Column

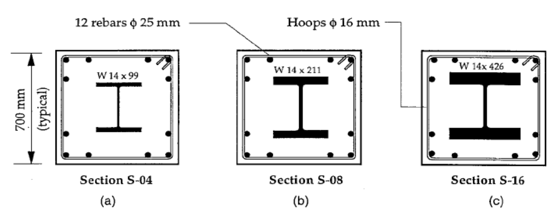

Fig.2. Prototype Composite Column: (a) S-04; (b) S-08; (c) S-16

For high seismic regions where large member ductility is required, the AISC/LRFD Seismic Provisions (…) for encased composite columns require transverse hoop reinforcement with a minimum area A_{sh} equal to

$$A_{sh} = 0.09 h_c s \left(1 - \frac{F_{ys} A_s}{P_n}\right) \left(\frac{f_c'}{F_{yh}}\right)$$

where h_c = cross-sectional dimension of the confined core region measured center-to-center of the tie reinforcement; s = vertical spacing of the hoop reinforcement; F_{ys} = specified yield strength of the structural steel; A_s = cross-sectional area of the structural steel core; P_n = nominal compressive axial strength of the composite column; f_c' = specified concrete compressive strength; and F_{yh} = specified yield strength of the ties. In subsequent analyses, the seismic hoop reinforcement shown in Fig. 3, is investigated 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.

Defining the yield $\varnothing_y$, and ultimate $\varnothing_u$ curvatures as shown in Fig. 4, the curvature ductility of the cross section is defined as

$$\mu_\varnothing = \frac{\varnothing_u}{\varnothing_y}$$

Members of frames designed for inelastic action in regions of high seismicity should have curvature ductilities of approximately $\mu_\varnothing > 12$. The inelastic behaviour of the S-08 composite cross sections with three concrete strength were evaluated based on the moment versus curvature behaviour and shown in Fig. 5 - 9.

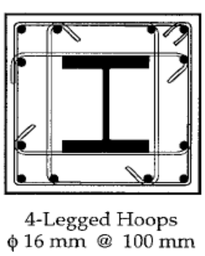

Fig. 3. Fiber Idealization of Encased Concrte Columns

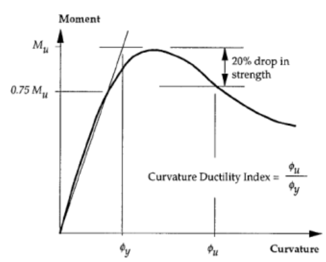

Fig. 4. Definition of Curvature Ductility Ratio

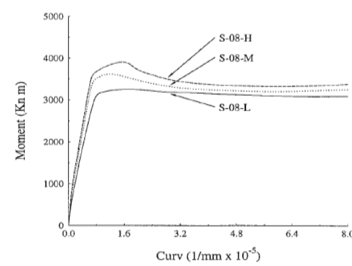

Fig. 5. Moment versus Curvature Response for Section S-08 as Function of Concrete Strength

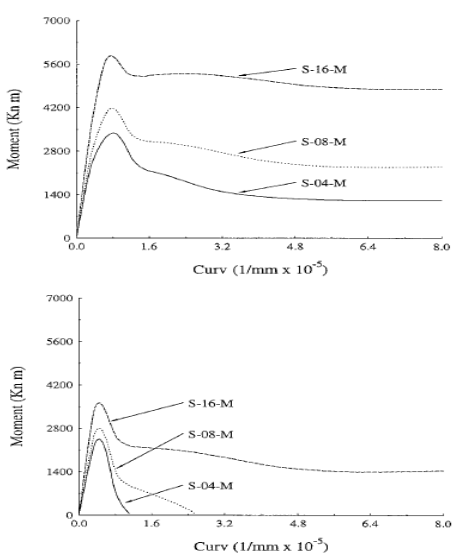

Fig. 6. Response of Sections with Medium-Strength Concrete: (a) $P = 0.3P_o$; (b) $P = 0.6P_o$

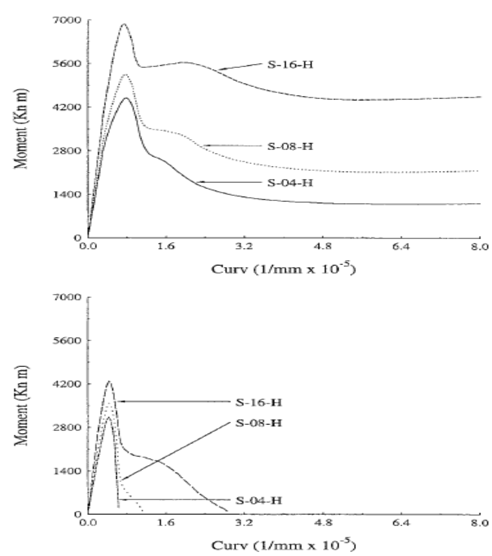

Fig. 7. Response of Sections with High-Strength Concrete: (a) $P = 0.3P_o$; (b) $P = 0.6P_o$

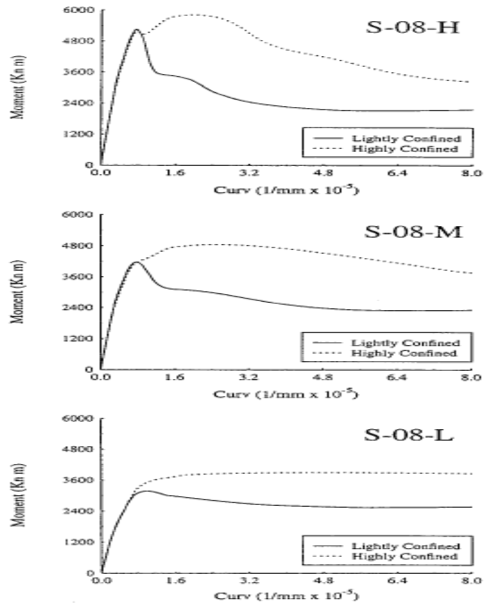

Fig. 8. Comparison of Responses of Section S-08 with Standard Ties and Seismic Hoop Reinforcement and $P = 0.3P_o$: (a) High-Strength Concrete; (b) Medium-Strength Concrete; (c) Low-Strength Concrete

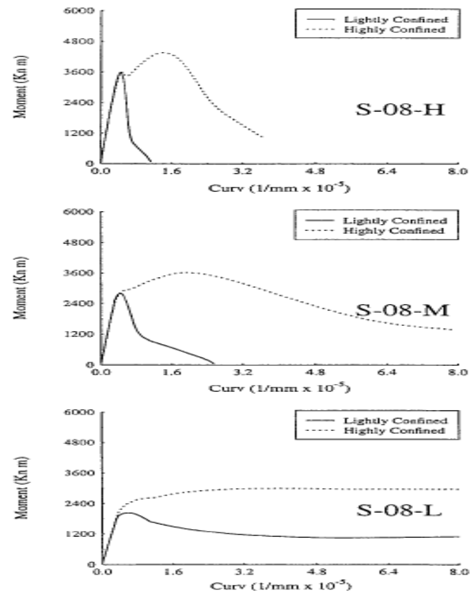

Fig. 9. Comparison for Responses of Section S-08 with Standard Ties and Seismic Hoop Reinforcement and $P = 0.6P_o$: (a) High-Strength Concrete; (b) Medium-Strength Concrete; (c) Low-Strength Concrete

The results of the evaluation of the 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/LRFD Seismic Provisions for composite columns.
3. The compression load $P = 0.6P_o$ 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 (Fig. 1**)**

Column Demands of Special Two-Story X-Braced Frames

Two-story X-braced frames was considered as a better configuration of Chevron frames(Hewit, AISC Seismic..) because the brace on the upper story brace in tension will resist the unbalance force on the beam (Fig. 10), allowing a smaller beam section to be used. The investigation done by Richards( ) shows that the

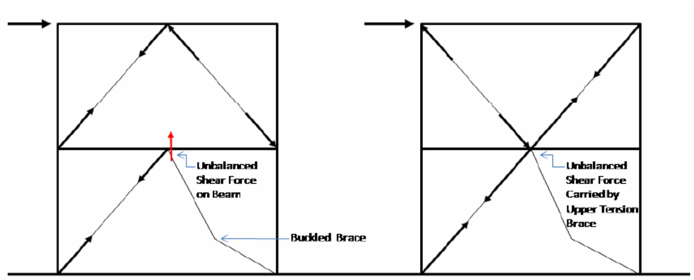

Fig. 10. Unbalanced VS Balanced Chevron Connections

column demands in two-story X-Braced frames were unrealistically high. The axial forces in the columns are sensitive to buckling of the braces as shown in Fig. 11. When brace is removed, analogous to buckling, column demands double even with the same floor forces. The 2005 AISC Seismic Provisions do not require columns to be designed for the maximum force that can be delivered to them if $\frac{\mathrm{kl}}{\mathrm{r}} \leq 4\sqrt{\frac{\mathrm{E}}{\mathrm{Fy}}}$. If $\frac{\mathrm{kl}}{\mathrm{r}} \leq 4\sqrt{\frac{\mathrm{E}}{\mathrm{Fy}}}$ columns are designed for loads corresponding to twice the axial loads caused by the design base shear (P_u = Ω_o x P_{base_shear}, where Ω_o = 2). This approach is based on engineering judgement that is need to be questioned for the SCBFs with braces in the two-story X-configuration. The design of the columns should be done based on the maximum load that can be delivered by the braces ( ).

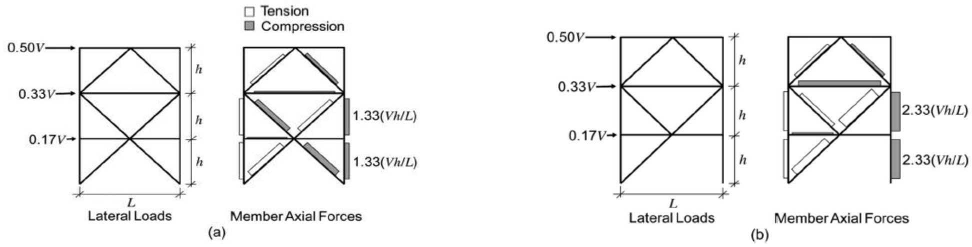

Fig. 11. Forces in SCBF with two-story X bracing: (a) before brace removal; (b) after brace removal

Composite Special Concentrically Two-Story X-Braced Frames

Encased composite columns can be an ideal solution for use in seismic regions. It is anticipated that the overall behaviour of the composite systems will be similar to SCBF counterpart and that inelastic deformations will occur through axial yielding and/or buckling of braces. There are two options for the design of the columns in SCBFs based on the slenderness of the braces (Aisc 341):

1. § 4.1 - AISC Seismic Provisions: for $\frac{K.L}{r} \leq 4\sqrt{\frac{E}{Fy}}$, then $P_u = \Omega_o$ x $P_{base\ shear}$, where $\Omega_o = 2$. This approach is based on engineering judgement that is need to be questioned for the SCBFs with braces in the two-story X-configuration.
2. §13.2a - AISC Seismic Provisions: for $4\sqrt{\frac{E}{Fy}} < \frac{K.L}{r} < 200$, then P_u equal to the bracing capacity. This is a more rational approach for the SCBFs with braces in the two-story X-configuration.

Fig. 12 shows the elevation view of the lateral resisting system of a five-story office building constructed at a hard soil in zone 6 region of Indonesia. All braced bays have the two-story-X configuration. W shapes (F_y = 350 MPa) are used for all beams and columns. Square HSS (F_y = 46 Ksi) are used as braces 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 are shown in Table 1.

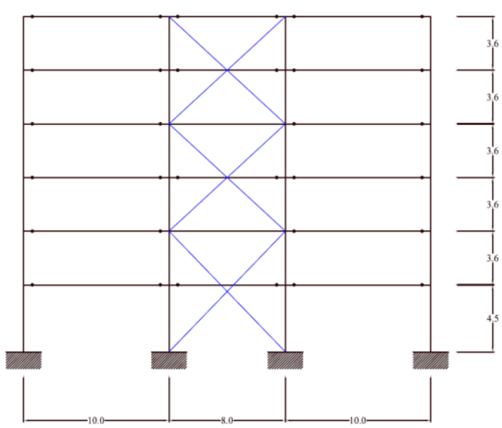

Fig. 12. Elevation View

Table 1. Bracing Capacities (kN)

HS178x178x10	+2744.41	-1983.19
HS178x178x11	+3152.97	-2296.33
HS203x203x13	+4112.18	-3264.75
HS203x203x13	+4112.18	-3264.75
HS203x203x13	+4112.18	-3264.75
HS203x203x13	4112.18	-2879.12

The column demands based on the bracing capacity are shown in Table 2. It can be seen the the column demands are unrealistically high and very heavy columns would be the results of the design if bare steel columns are employed.

Table 2. Maximum Possible Columns Demands (kN)

	Strength Design 1.2xD + 0.5xL± 1.0xE_h		***Ductility Design*** 1.2xD + 0.5xL± (bracing capacities)	
6	**230.71**	W200x46	**2094.98**	W200x100
5	**493.45**	W200x46	**2492.58**	W200x100
4	**1092.82**	W250x58	**7138.92**	W310x226
3	**1356.95**	W250x58	**7537.74**	W310x226
2	**2195.66**	W310x97	**12897.25**	W360x382
1	**2465.26**	W310x97	**13301.85**	W360x382

The composite special two-strory X braced frames is an economical solution to the high additional axial load capacity over that available with steel columns alone. Under severe seismic conditions the appropriate collapse-avoidance strategy could employ:

1. Composite columns with normal strength concrete (f_c' = 28 MPa)
2. Confinement steels in the form of transverse hoop reinforcement
3. Compression load $P \leq 0.6P_o$
4. Large steel cores, where the steel sections alone can resist the1.2xD + 0.5xL ± 1.0xE_h.

Design of composite columns are quite straightforward. Examples of the design can be seen in the AISC design examples CD.

Conclusions

Care should be taken when computing maximum possible columns demands in special frames with two-story X-braced configuration. In high seismic regions, concrete encased composite columns can be an ideal solution to deal with the additional axial load capacity over that available with steel columns alone. The composite system can be expected to have the overall behaviour similar to SCBFs counterpart.

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Fig. 12 shows the elevation view of the lateral resisting system of a five-story office building constructed at a hard soil in zone 6 region of Indonesia. All braced bays have the two-story-X configuration. The maximum columns loads are calculated using the following load combinations:

1. 1.2 x D + 1.6 x L
2. 1.2 x D + 0.5 x L ± 1.0 x E_h

W shapes (F_y = 350 MPa) are used for all beams and columns. Square HSS (F_y = 46 Ksi) are used as braces 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 are shown in Table 1. The column demands based on the bracing capacity are shown in Table 2. It can be seen the the column demands are unrealistically high and very heavy columns would be the results of the design if bare steel columns are employed.

The composite special two-strory X braced frames is an economical solution to the high additional axial load capacity over that available with steel columns alone. Under severe seismic conditions the appropriate collapse-avoidance strategy could employ:

5. Composite columns with normal strength concrete (f_c' = 28 MPa)
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