Design of Composite RCS Special Moment Frames

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This document was developed to assist practicing engineers in the design of Composite Special Moment Frame (C-SMF) systems utilizing reinforced concrete columns and steel beams (known as Composite RCS frames). These systems utilize the intrinsic advantages of each material which are optimized in resisting the applied loads.

Seismic design requirements for C-SMF systems are included in ASCE 7 (2010) and AISC 341 (2010). These system-level requirements are supported by research and other documents on the design and detailing of beam-column connections between the steel beams and concrete (or encased composite) columns. In 1994, the ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete issued guidelines for the Design of Joints between Steel Beams and Reinforced Concrete Columns in 1994 (ASCE 1994). Based on research at the time, it was recommended to limit the use of Composite RCS systems to regions of low seismicity. Since then, further research has been performed which has demonstrated that Composite RCS systems can be designed to have reliable ductile performance, making them an attractive design alternative for high seismic areas. Based on this research, a draft Pre-Standard for the Design of Moment Connections between Steel Beams and Concrete Columns (ASCE 2015 draft) has been prepared as an update to the 1994 ASCE connection design guidelines. This draft is utilized for the design studies presented herein.

**Definitions** For definitions not contained in this report, see the ASCE 7 (2010), AISC-341 (2010), ASCE Pre-Standard (2015 draft) and ACI-318 (2011).

This document is divided into three chapters and five appendices:

**Chapter 1** provides the basic design and acceptance guidelines presented within ASCE 7 (2010), AISC 341 (2010) and the ASCE Pre-Standard (ASCE 2015 draft). For information not contained in this document, please refer to the aforementioned publications.

**Chapter 2** illustrates the application of the basic design code provisions and connection design guidelines for composite frames to the design of a three-story long span warehouse building located in a high seismic zone. It should be noted that the code provides minimum design requirements and the calculations provided here are intended to show how these minimum provisions apply. It is beyond the scope of this document to comment on the adequacy of the code minimums in relation to any particular level of damage or risk.

**Chapter 3** illustrates the use of passive damping with composite systems. Viscous dampers are introduced to a conventional frame, and the use of nonlinear response history analysis (NLRHA) procedure is also illustrated.

**Appendix A** discusses nonlinear modeling using SAP2000. The non-linear model was used to validate the results of the full-scale composite test reported by Cordova and Deierlein (2005).

**Appendix B** describes the use of linear model using SAP 2000 to replicate results obtained from nonlinear modeling.
Appendix C illustrates the parametric study performed to discuss differences between linear and non-linear modeling of composite frames.

Appendix D consists of results of the cost study performed for a building using composite RCS frame and steel moment frame as its seismic force resisting systems.

Appendix E includes a preliminary draft of the updated design requirements for composite beam-column connections.
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### SYMBOLS AND NOTATIONS

\[ A_{sh} \] cross-sectional area of reinforcement parallel to beam (including cross ties) with spacing \( s_h \);

\[ A_{tie} \] total area of ties parallel to beam and located within 0.4\( d \) of beam;

\( a_c \) length of concrete bearing zone;

\( b \) width of concrete column measured perpendicular to beam;

\( b_f \) width of steel beam flanges;

\( b_i \) width of inner concrete panel;

\( b_j \) effective width of joint panel;

\( b_o \) effective width of outer concrete panel;

\( b_p \) width of FBP;

\( b'_p \) width of steel column or extended FBP;

\( C_c \) compression force in concrete bearing zone;

\( C_{cn} \) nominal compression strength of bearing zone;

\( C_{vr} \) compression force in vertical reinforcement;

\( C_{vrm} \) nominal compression strength of vertical reinforcement;

\( d \) depth of steel beam measured parallel to column;

\( d_b \) diameter of reinforcing bar;

\( d_{bp} \) depth of steel band plate measured parallel to column;

\( d_c \) steel column depth measured parallel to beam;

\( d_j \) effective joint depth;

\( d_o \) additional effective joint depth provided by attachments to beam flanges;

\( d_w \) distance between beam flanges (height of web);

\( F_{1,2} \) theoretical forces in vertical column bars above and below joint;

\( S_s \) mapped response acceleration, short periods;

\( S_1 \) mapped response acceleration, 1s period;

\( R \) Response Modification Unit;

\( C_t \) building period coefficient;

\( X \) period parameter;

\( F_a \) short period site coefficient;

\( F_v \) long period site coefficient;

\( S_{DS} \) design 5\% damped, spectral response acceleration at short periods;

\( S_{D1} \) design 5\% damped, spectral response acceleration at a period of 1 sec;
1. DESIGN AND ACCEPTANCE CRITERIA

1.1. Analysis and Modeling

1.1.1. Seismic Force

The seismic design base shear is calculated in accordance with ASCE 7.

1.1.2. Design Coefficients and Factors

Design coefficients and factors, such as $R$, $\Omega_0$, $C_d$, are those specified for special composite moment frames per ASCE 7. These values are the same as those used for special steel or concrete moment frames, equal to $R=8$; $\Omega_0=3$; $C_d=5.5$.

1.1.3. Story Drift Limit

The expected inelastic deflections are calculated by using elastic deflection and the amplification factor per ASCE 7. The design story drift ($\Delta$) should not exceed the allowable story drift ($\Delta a$) per ASCE 7.

1.1.4. Member Stiffness

1.1.4.1 Reinforced Concrete Columns

The stiffness of the reinforced concrete columns are defined to represent the conditions of an effective cracked section. A stiffness modification factor $= 0.7$ is recommended per ACI unless cracking is adequately modelled and evaluated using either testing or other analysis. The parametric study in Appendix C, illustrates one possible method of determining the stiffness modification factor.

1.1.4.2 Steel Beams

For the purpose of designing the lateral response of the special composite moment frame, the beams are analyzed and designed as bare steel beams. The strength and stiffness contribution from the composite slab is ignored in accordance with current design practice for steel special moment frames. Ignoring the composite action is conservative from the standpoint of establishing the required minimum beam strength (to satisfy the minimum base shear) and for minimum stiffness requirements (to satisfy the maximum drift limits). Since ignoring the strengthening effects of composite action may be unconservative for checking the strong-column weak-beam requirements, composite action is accounted for in these checks to avoid the formation of story mechanisms and excessive yielding in the columns.

1.1.4.3 Connections

Joint deformations should be considered in the frame design. The following are a few suggested approaches, based on methods commonly employed for steel and concrete moment frames:

1. Neglecting finite joint size and computing member stiffness based on centerline dimensions.
2. Employing a modified finite joint dimension (i.e. 50% rigid joint) in kinematic constraints applied to the beams and columns or in adjustments to the beam and column stiffness.
3. Using a combination of rigid end offsets and a rotational spring between the beam and column at the joint to account for finite joint size and flexibility.
While the use of centerline dimensions may result in conservative predictions of elastic drifts, this is generally accepted in standard design practice. Other methods to account for joint stiffness should be based on engineering judgment informed by relevant test data and mechanics-based models.

### 1.2. System Requirements

#### 1.2.1. Moment Ratio

Section G3.4a of AISC 341 should be satisfied for the Strong Column and Weak Beam (SCWB) check at beam-to-column connections. In this check, the beam strength should be calculated as a composite beam, where beams in positive bending should have the full composite strength and a beam in negative bending should have the expected plastic moment of the bare steel beam.

**AISC 341 G3.4a. Moment Ratio**

The following relationship shall be satisfied at beam-to-column connections.

\[
\frac{\sum M_{pcc}^*}{\sum M_{p,exp}} > 1.0
\]

Where:
- \( \sum M_{pcc}^* \) = Sum of the moments in the columns above and below the joint at the intersection of the beam and column centerlines. \( \sum M_{pcc}^* \) is determined by summing the projections of the nominal flexural strengths, \( M_{pcc} \), of the columns above and below the joint to the beam. For reinforced concrete columns, the nominal flexural strengths, \( M_{pcc} \), shall be calculated based the provisions of ACI 318 with consideration of the required axial strength.
- \( \sum M_{p,exp} \) = Sum of the moments in the steel beams at the intersection of the beam and column centerlines. \( M_{p,exp}^* \) is determined by summing the expected flexural strengths of the beam at the plastic hinge locations the column centerline. It is permitted to take \( \sum M_{p,exp}^* = \sum (1.1M_{p,exp} + M_{uv}) \), where \( M_{p,exp} \) is the expected flexural strength of the steel beams or composite beams.
- \( M_{uv} \) = Moment due to shear amplification from the location of the plastic hinge to the column centerline.

#### 1.2.2. Stability Bracing of Beams

Beams should be braced to satisfy the requirements for highly ductile members in Section D1.2b of AISC 341.

**AISC 341 D1.2b. Highly Ductile Members**

In addition to the requirements of Sections D1.2a(a)(1) and (2), and D1.2a(b)(1) and (2), the bracing of highly ductile beam members shall have a maximum spacing of \( L_b = 0.086r_y E / F_y \). For concrete-encased composite beams, the material properties of the steel section shall be used and the calculation for \( r_y \) in the plane of buckling shall be based on the elastic transformed section.

#### 1.2.3. Stability Bracing at Beam-to-Column Connections

In most buildings, the beam-to-column connection in composite frames can be considered as braced by the composite slab floor diaphragm. Where a floor diaphragm is not present around the connections, then composite columns with unbraced connections should satisfy the requirements of Section E3.4c (2) of AISC 341.
1.3. Member Requirements

1.3.1. Basic Requirements

Steel and composite members should satisfy the requirements of Sections D1.1 of AISC 341 for highly ductile members.

The design of the reinforced concrete columns in a composite moment frame should be in accordance with provisions of ACI 318 for concrete columns in seismic special moment frames.

**AISC 341 D1. Member Requirements**

*Members of moment frames, braced frames and shear walls in the seismic force resisting system (SFRS) shall comply with the Specification and this section. Certain members of the SFRS that are expected to undergo inelastic deformation under the design earthquake are designated in these provisions as moderately ductile members or highly ductile members.*

**ACI 318 21.6 Special Moment Frame Members Subjected to Bending and Axial Load**

*Requirement of this section apply to special moment frame members that form part of seismic-force-resisting system and that resist a factored axial compressive force Pu under any load combination exceeding \( A_{g}'c/10 \). These frame members shall also satisfy the condition of 21.6.1.1 and 21.6.1.2.*

1.3.2. Beam Flanges

The beam flanges should satisfy the requirements of AISC 341 Section G3.5b.

**AISC 341 G3.5b. Beam Flanges**

*Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is prohibited unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle.*

1.3.3. Protected Zones

The region at each end of the beam subject to inelastic straining should be designated as a protected zone and satisfy the requirements of Section D1.3 of AISC 341. The protected zone can be considered as extending from the face of concrete column to one half of the steel beam depth beyond the plastic hinge point. The plastic hinge point for the C-SMF is at the face of the column.

**AISC 341-10 D1.3. Protected Zones**

*Discontinuities specified in Section I2.1 resulting from fabrication and erection procedures and from other attachments are prohibited in the area of a member or a connection element designated as a protected zone by these Provisions or ANSI/AISC 358.*

1.4. Connections

Connections are considered as fully restrained (FR) and should satisfy the requirements of AISC 341 Section D2 and this section.
1.4.1. Beam-to-Column Connections

In the composite RCS frames, the steel beam runs continuous through the column, and the field splice is sufficiently far enough away from the column that there are no critical welded or bolted joints within the plastic hinge region. The connections are designed as fully restrained connections using the ASCE Pre-Standard (2015 draft) to develop the full beam strength. Due to the through-beam connection configuration, no continuity or diaphragm plates are needed for the composite RCS connections to either concrete or concrete encased steel columns. Therefore, the composite connections to highly ductile steel beams are deemed sufficiently ductile to provide the required ductility given by the story drift angle of 0.04 rad. The conformance demonstration criteria fall into category (b) of AISC 341 G3.6c, where their design is based on “other substantiating data”, which consists of the calibration and validation done in developing the ASCE Pre Standard (2015 draft).

AISC 341 G3.6b. Beam-to-Column Connections

Steel Beam-to-Concrete Column connections used in the SFRS shall satisfy the following requirements:

1. The connection shall be capable of accommodating a story drift angle of at least 0.04 rad.
2. The measured flexural resistance of the connection, determined at the column face, shall equal at least 0.80 $M_p$ of the connected beam at a story drift angle of 0.04 rad, where $M_p$ is defined and calculated as in Section G2.6b.

$M_p$ is defined as the nominal flexural strength of the steel, concrete-encased or composite beams and shall satisfy the requirements of Specification Chapter I.

AISC 341 G3.6c. Conformance Demonstration

Beam-to-column connections used in the SFRS shall satisfy the requirements of Section G3.6b by the following:

(a) When beams are interrupted at the connection, the connections shall be qualified using test results obtained in accordance with Section K2. Results of at least two cyclic connection tests shall be provided, and shall be based on one of the following:
   i. Tests reported in research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Section K2.
   ii. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified by Section K2.

(b) When beams are uninterrupted or continuous through the composite or reinforced concrete column, beam flange welded joints are not used, and the connection is not otherwise susceptible to premature fracture, the performance requirements of Section G3.6b shall be demonstrated in accordance with (a) or other substantiating data.

Connections that accommodate the required story drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified in Section G3.6d are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.
1.4.2. Joint Strength (per ASCE Pre-Standard, 2015 draft)

1.4.2.1. Shear Strength of the Connection

The shear strength of the connection should satisfy the following equation,

\[ V_j \leq k (\phi V_{in} + \phi_m V_{on}) \]  

(ASCE Pre-Standard Eq10)

where \( k \) is an adjustment factor to control connection deformations (\( k=1.0 \) where large connection deformations are considered in the design, \( k=0.85 \) where the intent is to limit joint deformations, and \( k \) shall be taken as 1.0 as long as joint is stronger than the strength of connected beams per capacity design), \( \phi_s = 0.85 \) for shear in the outer panel, and \( \phi \) for the inner panel is specified depending on the governing design check in ASCE Pre-Standard Section 3.2.1. Where the frame is modeled using centerline dimensions and the connection is designed to develop the full beam strength, it is considered reasonable to use \( k=1.0 \).

The required connection strength, \( V_j \), in (ASCE Pre-Standard Eq10) is calculated as,

\[ V_j = \sum \frac{M_{b}}{d_j} - V_c \]  

(ASCE Pre-Standard Eq11)

where \( \sum M_b \) and \( V_c \) are as defined in ASCE Pre-Standard Section 2.2, and \( d_j \) is the effective joint depth, determined as the distance between steel beam flange centerlines.

1.4.2.2. Inner Panel

The design strength of the inner panel, \( V_{in} \), is governed by either the combined horizontal shear strengths of the steel web panel, \( V_{spn} \), and inner diagonal concrete strut, \( V_{icn} \), or by the vertical bearing capacity of the inner panel. Thus,

\[ \phi V_{in} = \phi_s (V_{sp} + V_{ic}) \leq \phi_b \left( \frac{M_{vb} - V_k b}{d_j} \right) \]  

(ASCE Pre-Standard Eq12)

where \( V_{sp}, V_{ic}, \) and \( M_{vb} \) are determined by Eq. 13 - 15, \( \phi_b = 0.85 \), and \( \phi_b = 0.75 \).

The horizontal shear strength of the steel web panel and the inner diagonal concrete strut are determined as,

\[ V_{sp} = 0.6 F_{ysp} t_{sp} \alpha_{sp} h \]  

(ASCE Pre-Standard Eq13)

\[ V_{ic} = 1.7 \alpha_c \sqrt{f_{c'}} b_j h \leq 0.5 f_{c'} b_j d_j \]  

(ASCE Pre-Standard Eq14)

where \( F_{ysp} \) and \( t_{sp} \) are the yield strength and thickness of the steel panel, respectively, \( \alpha_{sp} = 0.9 \) and 0.8 for interior and exterior connections, respectively, \( \alpha_c = 1.0 \) and 0.6 for interior and exterior connections, and \( f_{c'} \) is the concrete compressive strength in MPa.

The moment bearing strength of the connection, \( M_{vb} \), is determined as,
\[ M_{vb} = C_{cn} \frac{h}{2} \left( 1 - \beta_1/2 \right) + h_v r (T_{vrn} + C_{vrn}) \]  
(ASCE Pre-Standard Eq15)

\[ C_{cn} = \alpha_{cn} f'c b_f (a_1 h/2) \]  
(ASCE Pre-Standard Eq16)

Where \( T_{vrn} \) and \( C_{vrn} \) = the nominal strengths in tension and compression, respectively, of the vertical joint reinforcement, which is attached directly to the steel beam, and \( h_vr \) = the distance between the bars. For concrete strengths up to and including 4ksi (27.6 MPa) the factor \( \alpha_1 = 0.85 \), and for strengths greater than 8ksi (55.2 MPa) \( \alpha_1 = 0.65 \). For concrete strengths between these limits, \( \alpha_1 \) should be linearly interpolated. For bearing regions with minimum reinforcing bar ties, \( \alpha_{cn} = 2.0 \); and for bearing regions confined by band plates, \( \alpha_{cn} = 2.5 \).

The following factors should be considered in the strength calculation for \( T_{vrn} \) and \( C_{vrn} \): connection between the reinforcement and steel beam, development of the reinforcement through bond or anchorage to concrete, and the material strength of the reinforcement. In addition, for use in (ASCE Pre-Standard Eq15), the contribution of the vertical reinforcement is limited as follows:

\[ T_{vrn} + C_{vrn} \leq C_{cn} / 2 \]  
(ASCE Pre-Standard Eq17)

1.4.2.3. Outer Panel

The nominal strength of the outer diagonal concrete strut, \( V_{on} \), should be determined as:

\[ V_{on} = 1.25 \alpha_x \sqrt{f_{c,b} t_{bp}} h \]  
(ASCE Pre-Standard Eq18)

\[ b_x = y + \frac{2}{3} \alpha_x x - b_f \leq b - b_f \]  
(ASCE Pre-Standard Eq9)

where \( \alpha_x \) is as defined previously for the inner panel (Eq14), \( x \) and \( y \) are effective dimensions of the shear keys, and \( t_{bp} \) is the thickness of the band plate. Where the column is subjected to a net tension force larger than 0.1f'cAg under the design loads, then \( V_{on} = 0 \). Hoops within the beam depth as well as adjacent to the joint, and steel band plates should satisfy the requirements given in Section 1-4-4. The concrete column width measured perpendicular to the beam = \( b \), \( y = \) the steel column or extended FBP width or \( y = b_f \) for the band plate. For extended FBPs and band plates, \( x = h \), and for encased steel columns, \( x = h/2 + d_c/2 \) where \( d_c \) = the steel column depth. When more than one shear key exists, the outer panel width should be determined based on the shear key that provides the largest width. If no shear keys are present, the outer panel width may be calculated using \( x = 0.7h \) and \( y = 0 \), based on friction resistance between the beam and concrete in the vertical bearing region. Where steel band plates are used in lieu of reinforcing bar ties in the bearing region above and below the beam, \( \alpha_x = 0.5 \); otherwise, where ties are present (with or without the band plate), \( \alpha_x = 1.0 \).

1.4.3. Detailing Considerations (per ASCE Pre-Standard, 2015 draft)

1.4.3.1. Confinement Requirements within the Beam Depth

Concrete within the beam depth shall be confined by horizontal reinforcing bars or other means, such as steel cover plates. Where horizontal reinforcing bar ties are used, they should be provided in the column within the beam depth with a minimum volumetric ratio of \( \rho_f = 0.01 \), where \( \rho_f \) is the ratio between the tie volume within the beam depth and the joint volume (\( d_f h b \)). The maximum spacing between horizontal ties (\( s_i \)) should not exceed the smaller of 0.25d_f and 0.25h.
Perimeter ties and cross ties may be developed by either 90° hooks, which engage a longitudinal bar, or by lap splicing the ties. For connections with beams framing in two orthogonal directions, the steel beams with FBPs may be considered as providing confinement within the joint depth. In such cases, steel ties, anchored in the inner concrete panel, should be used to provide support to the longitudinal column bars. Hook details and splice lengths should conform to the provisions of ACI 318.

### 1.4.3.2. Concrete Confinement Adjacent to Connection

Concrete confinement in the column above and below the beam should be provided by reinforcing bar ties or steel band plates. Where band plates are used, the band should extend at least 0.25dj above and below the beam and should be attached to the beam flanges and stiffened with a vertical stiffener plate. The steel band plates should have a minimum volume of $0.03 \frac{d_j h b}{h}$ and a minimum thickness of 0.5 in (13 mm). For joints without band plates, a minimum of three layers of ties should be provided above and below the beam, and the bars in each layer should be at least equivalent to the following:

- For $b \leq 20$ in (500 mm): four #3 (10-mm) bars
- For $20$ in (500 mm) < $b \leq 30$ in (750 mm): four #4 (13-mm) bars
- For $b > 30$ in (750 mm): four #5 (16-mm) bars

These ties should be closed rectangular ties that can resist tension parallel and perpendicular to the beam. For seismic design, the ties should satisfy seismic hoop requirements of ACI 318. The three layers should be located within a distance of $0.4d_j$ above and below the beam.

When band plates, extended FBPs or embedded steel columns are used to activate the outer concrete strut, the reinforcing bar ties or band plate above and below the beam may be governed by the need to transfer the force $V_{on}$ from the beam flanges to the outer concrete panel. The minimum total cross-sectional area $A$ of the ties or band plate should meet the following

$$A \geq \frac{V_{on}}{F_y}$$  \hspace{1cm} \text{(ASCE Pre-Standard Eq19)}

where $V_{on}$ is the nominal horizontal shear strength of the outer concrete strut, and $F_y$ is the yield strength of the reinforcement or band plate. The calculated area, $A$, is the total cross-sectional area (measured through a vertical plane perpendicular to the beam) of the band plate or ties located within a vertical distance $0.4d_j$ above and below the steel beam.

### 1.4.3.3. Longitudinal Column Reinforcing Bars

The size of vertical column bars passing through the joint should be limited as follows:

$$d_b \leq \frac{d_j (420)}{20 F_{yr}} \leq \frac{d_j}{20}$$  \hspace{1cm} \text{(ASCE Pre-Standard Eq20)}

where, for single bars, $d_b =$ vertical bar diameter, and for bundled bars, $d_b =$ the diameter of a bar of equivalent area to the bundle, and $F_{yr} =$ yield strength of longitudinal column bars in MPa. When steel band plates are used, the joint depth may be assumed equal to $1.25d_j$. 

\[ \rho_s \geq 0.01 \]
\[ s_h \leq \min(0.25d_j, 0.25h) \]
1.4.3.4. Steel Beam Flanges

The vertical bearing force associated with joint shear in the steel panel causes bending of the steel beam flanges. The beam flanges can be assumed capable of resisting transverse bending if the thickness satisfies the following:

\[
t_f \geq 0.30 \sqrt{\frac{b_f \cdot t_{sp} \cdot d \cdot F_{ysp}}{h \cdot F_{yf}}}
\]

(ASCE Pre-Standard Eq21)

where \( t_{sp} \) and \( F_{ysp} \) = the thickness and yield strength of the steel panel, respectively, and \( F_{yf} \) = the yield strength of the beam flanges.

1.4.3.5. Face Bearing Plates within Beam Depth

The FBPs within the beam depth should be detailed to resist a horizontal shear force equal to the horizontal nominal shear strength of the inner concrete strut, \( V_{icn} \). The FBP thickness, \( t_p \), should meet the following conditions,

\[
t_p \geq \sqrt[3]{\frac{V_{icn}}{2b_f F_{yp}}}
\]

(ASCE Pre-Standard Eq22)

\[
t_p \geq 0.20 \sqrt{\frac{V_{icn}b_f}{F_{yp}d_w}}
\]

(ASCE Pre-Standard Eq23)

where \( F_{yp} \) and \( F_{up} \) = the specified yield strength and tensile strength of the bearing plate, respectively. Additionally, the FBP thickness \( t_p \) should be such that,

\[
t_p \geq b_f/22
\]

(ASCE Pre-Standard Eq24)

1.4.3.6. Bearing Resistance of Steel Band Plates, Extended Face Bearing Plates and Steel Column

Where used, the band plates, extended FBPs, and/or steel column should be designed to resist a force equal to the joint shear carried by the outer compression strut, \( V_{on} \). The average concrete bearing stress against these elements should be less than or equal to \( 2f_c' \) and should be considered to act over a height not exceeding \( 0.25d_j \).

The minimum thickness of the column flanges, band plate, or extended FBPs should satisfy,

\[
t \geq 0.12 \sqrt{\frac{V_{on}b_p'}{d_{bp} F_{ybp}}}
\]

(ASCE Pre-Standard Eq25)

\[
t \geq \frac{\sqrt{3}}{2 b_p F_{ubp}} V_{on}
\]

(ASCE Pre-Standard Eq26)

where \( d_{bp} \) = depth of steel band plate \((0.25d_j)\); \( F_{ybp} \) and \( F_{ubp} \) = the specified yield strength and tensile strength of the steel band plates, respectively.
2. DESIGN EXAMPLE OF COMPOSITE RCS SPECIAL MOMENT FRAME

2.1. General Information

2.1.1. Narrative

A 54,000 square foot, three-story long-span warehouse building is considered for this design example. The roof framing of the warehouse consists of bare metal deck supported by steel beams, girders and concrete columns. The framing of the 2nd and 3rd floors consists of concrete over metal deck supported by steel beams, girders and concrete columns. The foundation system consists of isolated pad footing connected with grade beam. Design of the foundation system is beyond the scope of this design example.

Figures 2-1 through 2-5 illustrate the geometry of the building, and Table 2-1 summarizes the structural framing members. Later figures in the chapter include connection details and schematic construction sequences.

The seismic force resisting system (SFRS) consists of Composite Special Moment Resisting Frames with concrete columns and steel beams. These frames are laid out in a balanced manner to avoid torsional irregularity. Based on the governing load case for the beam design, and to simplify the design example, pin connection is chosen at the end of frames in the long direction of the building. This choice of pin connections at the end is not intended to discourage the use of two-way moment systems. Two-way systems should be considered in the design of buildings where applicable, such as short span building with similar bays in both directions.

Similar to a steel moment frame system, the composite RCS system member sizes are governed by the drift performance requirements. In the design example considered, the concrete column size is governed by the strong column weak beam requirements, which is expected to be the case for a typical moment frame system.

The expected yielding locations for the SFRS are at the ends of the steel beams and at the bases of the concrete columns. Stability of the steel beams and the concrete columns is required to achieve this desired behavior. Therefore, the connection detail between the steel beam and the concrete column is the key to the RCS system. It is also important to understand the erection/construction sequence and the method of concrete construction being considered for the project. These may vary based on the location of the project, capabilities of the construction contractor, and the size of the project. Two possible erection sequences are illustrated in this design example to aid the engineers. It is recommended that the engineer discuss the potential erection and construction sequence with the contractor for their project before or while performing the design of the system.

This following design steps are included in the example.

1) Load Criteria & Load Combination
2) Seismic Force Calculation including \( R, \Omega_0 \) and \( C_d \) assignment
3) Creating Analysis Model
4) Seismic Design Requirements Checks
5) Member & Connection Design
6) Typical Details and Construction Procedures
2.1.2. Geometry Definition

Figure 2-1. Typical Floor Framing Plan

Figure 2-2. Foundation Floor Framing Plan
**Structural Members**

The structural framing member sizes are summarized in Table 2-1.

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Section</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bx, GY</td>
<td>RF</td>
<td>W21x62</td>
<td>Moment Frame Beam</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>W30x108</td>
<td>Moment Frame Beam</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>W30x108</td>
<td>Moment Frame Beam</td>
</tr>
<tr>
<td>B1</td>
<td>RF</td>
<td>W16x26</td>
<td>Gravity Beam</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>W24x68</td>
<td>Gravity Beam</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>W24x68</td>
<td>Gravity Beam</td>
</tr>
<tr>
<td>G1</td>
<td>RF</td>
<td>W21x50</td>
<td>Gravity Beam</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>W30x108</td>
<td>Gravity Beam</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>W30x108</td>
<td>Gravity Beam</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Section</th>
<th>Reinforcing</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>GBx</td>
<td>1st</td>
<td>32&quot; x 40&quot; (813x1016)</td>
<td>(6)#10 T&amp;B</td>
<td>#4 ties at 8&quot; OC</td>
</tr>
<tr>
<td>GBx</td>
<td>1st</td>
<td>24&quot; x 40&quot; (610x1016)</td>
<td>(6)#10 T&amp;B</td>
<td>#4 ties at 8&quot; OC</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Section</th>
<th>Reinforcing</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>3rd</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(20)#10</td>
<td>#4 ties at 6&quot; o.c. (4legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd-1</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(20)#11</td>
<td>#5 ties at 4&quot; o.c. (4legs each way)</td>
</tr>
<tr>
<td>C2</td>
<td>3rd</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(20)#10</td>
<td>#4 ties at 6&quot; o.c. (4legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd-1</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(20)#11</td>
<td>#5 ties at 4&quot; o.c. (4legs each way)</td>
</tr>
<tr>
<td>C3</td>
<td>3rd</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(16)#10</td>
<td>#4 ties at 6&quot; o.c. (4legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd-1</td>
<td>32&quot; x 32&quot; (813x813)</td>
<td>(16)#11</td>
<td>#5 ties at 4&quot; o.c. (4legs each way)</td>
</tr>
</tbody>
</table>

1: Lo Zone (ACI318 21.6.4.3)

Material Specifications:
(a) Steel Frame: A992, \( f_y = 50 \text{ ksi} \)
(b) High-strength bolts: A325 S.C.
(c) Welding electrodes: E70
(d) Concrete strength: \( f'_c = 6,000 \text{ psi} \)
(e) Rebar: Gr. 60
2.1.3. Code and Criteria

(1) The building is designed in accordance with the 2012 International Building Code (IBC) and ASCE 7-10 (ASCE 7).

(2) Design of steel members and connection is based on AISC 360-10 (AISC 360), AISC 341-10 (AISC 341), Blume Earthquake Engineering Center Report No.155 (Blume Center Report) and ASCE Pre-Standard given documents for connections between steel beams and reinforced concrete.

(3) Design of concrete members is based on ACI 318-11 (ACI 318).

(4) The Risk Category is classified as II, per ASCE 7.

(5) The building is designed for Life Safety Performance Objective at DBE level seismic forces with Composite Special Moment Frames.

(6) Design coefficients and factors are as follows (ASCE 7 Table 12.2-1):
- Response Modification Coefficient: \( R = 8 \)
- Overstrength Factor: \( \Omega_0 = 3 \)
- Deflection Amplification Factor: \( C_d = 5.5 \)

(7) Story Drift Limit (ASCE 7, Table 12.12-1)
\[ \Delta_s \leq 0.025h_{sx} \]

2.1.4. Gravity Load Criteria

**Roof Loading**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>18 Ga. B Deck</td>
<td>3.0</td>
</tr>
<tr>
<td>Roofing/Insulation</td>
<td>6.0</td>
</tr>
<tr>
<td>Sprinklers</td>
<td>1.5</td>
</tr>
<tr>
<td>MEP</td>
<td>5.0</td>
</tr>
<tr>
<td>MISC</td>
<td>4.5</td>
</tr>
<tr>
<td>Superimposed Dead Load</td>
<td>20.0</td>
</tr>
<tr>
<td>Steel Beam</td>
<td>8.0</td>
</tr>
<tr>
<td>Concrete Column</td>
<td>20.0</td>
</tr>
<tr>
<td>Dead Load</td>
<td>28.0</td>
</tr>
<tr>
<td>Seismic Dead Load (D+SD)</td>
<td>48.0</td>
</tr>
<tr>
<td>Live Load (reducible)</td>
<td>20.0</td>
</tr>
</tbody>
</table>
### Floor Loading

- Flooring: 5.0
- Sprinkler: 1.5
- MEP: 5.0
- MISC: 7.5

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
<th>Unit</th>
<th>Conversion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superimposed Dead Load</td>
<td>19.0</td>
<td>psf</td>
<td>(910 N/m²)</td>
</tr>
<tr>
<td>Concrete Slab (3 ½&quot; NWC) w/ 18 Ga W3 Deck</td>
<td>64.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel Beam</td>
<td>20.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Column</td>
<td>20.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead Load</td>
<td>104.0</td>
<td>psf</td>
<td>(4980 N/m²)</td>
</tr>
<tr>
<td>Seismic Dead Load (D+SD)</td>
<td>123.0</td>
<td>psf</td>
<td>(5890 N/m²)</td>
</tr>
<tr>
<td>Live Load (warehouse light)</td>
<td>125.0</td>
<td>psf</td>
<td>(5985 N/m²)</td>
</tr>
</tbody>
</table>

### Perimeter Walls

- 200 lb/ft (2920 N/m)

#### 2.1.5. Building Mass and Weight

Per ASCE 7, Section 12.7.2, the effective seismic mass is to include a minimum of 25% of floor live load in areas used for storage. Considering the warehouse had high levels of occupancy in densely populated regions, 50% of floor live load was included as part of seismic load in this design example. Table 2-2 summarizes the building mass and weight. The **Total Building Weight (W) = 5650 Kips.**

<table>
<thead>
<tr>
<th>Level</th>
<th>Mi(Kip/(in/s²))</th>
<th>ΣMi(Kip/(in/s²))</th>
<th>Wi(Kip)</th>
<th>ΣWi(Kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>1.627</td>
<td>1.627</td>
<td>628.0</td>
<td>628.0</td>
</tr>
<tr>
<td>3rd</td>
<td>6.504</td>
<td>8.130</td>
<td>2511.0</td>
<td>3139.0</td>
</tr>
<tr>
<td>2nd</td>
<td>6.504</td>
<td>14.634</td>
<td>2511.0</td>
<td>5650.0</td>
</tr>
</tbody>
</table>

#### 2.1.6. Load Combination

The load combinations used in this design example are noted below. They are derived from Chapter 2 of ASCE 7. In this design example, the dead load is considered to consist of two components, Superimposed Dead Load (SD) which does not include self-weight of structure, and Dead Load (D) representing self-weight of the structure only.
(1) 1.4((D+SD)+F)
(2) 1.2((D+SD)+F+T)+1.6(L+H)+0.5(Lr or S or R)
(3) 1.2(D+SD)+1.6(Lr or S or R)+ (0.5L or 0.8W)
(4) 1.2(D+SD)+1.6W+L+0.5(Lr or S or R)
(5) 1.2(D+SD)+1.0E+L+0.2S  
     (1.2+0.2S) (D+SD)+ρE+L+0.2S
(6) 0.9(D+SD)+1.6W+1.6H
(7) 0.9(D+SD)+1.0E+1.6H  
     (0.9-0.2S) (D+SD)+pE+1.6H

2.2. Structural and Seismic Design Parameters

2.2.1. Structural and Site Information

The following building and site specific information was considered in this design example.

**Building and Site Specific Information**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped Response Acceleration, short periods $S_s$</td>
<td>2.170</td>
</tr>
<tr>
<td>Mapped Response Acceleration, 1s period $S_1$</td>
<td>0.743</td>
</tr>
<tr>
<td>Response Modification Coefficient $R$</td>
<td>8.0</td>
</tr>
<tr>
<td>Building Period Coefficient $C_t$</td>
<td>0.028</td>
</tr>
<tr>
<td>Period Parameter $X$</td>
<td>0.80</td>
</tr>
</tbody>
</table>

**Design Spectral Acceleration Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short Period Site Coefficient $F_a$</td>
<td>1.0</td>
</tr>
<tr>
<td>Long Period Site Coefficient $F_v$</td>
<td>1.5</td>
</tr>
<tr>
<td>Design 5% Damped, Spectral Response Acceleration at short periods $S_{DS}$</td>
<td>1.447</td>
</tr>
<tr>
<td>Design 5% Damped, Spectral Response Acceleration at a period of 1 sec $S_{D1}$</td>
<td>0.743</td>
</tr>
</tbody>
</table>

**Design Period**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approx. Fundamental Period $T_a$</td>
<td>0.59</td>
</tr>
<tr>
<td>Fundamental Period from Analysis $T_b$</td>
<td>0.97</td>
</tr>
<tr>
<td>$T_b$ (X)</td>
<td>0.97</td>
</tr>
<tr>
<td>$T_b$ (Y)</td>
<td>0.85</td>
</tr>
<tr>
<td>Long Period Transition $T_L$</td>
<td>8</td>
</tr>
</tbody>
</table>
2.2.2. Static Design Base Shear

**X-DIRECTION**

![Diagram of X-direction forces](image)

Seismic Forces (ASCE 7-10)

**Site parameters**
- Site class: D
- Mapped acceleration parameters (Section 11.4.1)
  - at short period: $S_s = 2.17$
  - at 1 sec period: $S_1 = 0.74$
- Site coefficient at short period (Table 11.4-1): $F_a = 1.0$
- at 1 sec period (Table 11.4-2): $F_v = 1.5$

**Spectral response acceleration parameters**
- at short period (Eq. 11.4-1): $S_{MS} = F_a \times S_s = 2.17$
- at 1 sec period (Eq. 11.4-2): $S_{M1} = F_v \times S_1 = 1.11$

**Design spectral acceleration parameters (Section 11.4.4)**
- at short period (Eq. 11.4-3): $S_{DS} = \frac{2}{3} \times S_{MS} = 1.45$
- at 1 sec period (Eq. 11.4-4): $S_{D1} = \frac{2}{3} \times S_{M1} = 0.74$

**Seismic design category**
- Risk category (Table 1.5-1): II
- Seismic design category based on short period response acceleration (Table 11.6-1): D
- Seismic design category based on 1 sec period response acceleration (Table 11.6-2): D
- Seismic design category: D
**Calculated fundamental period**

Height above base to highest level of building $h_n = 45$ ft

From Table 12.8-2:

- **Structure type** Steel moment frame
- **Building period parameter $C_t$** $C_t = 0.028$
- **Building period parameter $x$** $x = 0.80$
- **Coefficient for upper limit on calculated period (Table 12.8-1)** $C_U = 1.4$
- **Specified fundamental period** $T_{analysis} = 0.97$ sec
- **Approximate fundamental period (Eq 12.8-7)** $T_a = C_t \times (h_n)^x \times 1$ sec / (1ft)$^x = 0.59$ sec
- **Calculated fundamental period (Sect 12.8.2)** $T = \min(T_{analysis}, C_U \times T_a) = 0.82$ sec = $T_{used}$
- **Long-period transition period** $T_L = 8$ sec

**Seismic response coefficient**

- **Seismic force-resisting system (Table 12.2-1, C, 8)**
- **Response modification factor (Table 12.2-1)** $R = 8$
- **Seismic importance factor (Table 1.5-2)** $I_e = 1.0$
- **Seismic response coefficient (Sect 12.8.1.1)**
  - **Calculated (Eq 12.8-2)** $C_{s,\text{calc}} = \frac{S_{DS}}{(R / I_e)} = 0.181$
  - **Maximum (Eq 12.8-3)** $C_{s,\text{max}} = \frac{S_{D1}}{(T \times (R / I_e))} = 0.113$
  - **Minimum:**
    - Eq 12.8-5 $C_{s,\text{min}} = \max(0.044 \times S_{DS} \times I_e, 0.01) = 0.064$
    - Eq 12.8-6 (where $S_1 >= 0.6$) $C_{s,\text{min2}} = \frac{(0.5 \times S_1)}{(R / I_e)} = 0.046$
    - $C_{s,\text{min}} = 0.064$
  - **Seismic response coefficient** $C_s = 0.113$

**Seismic base shear (Sect 12.8.1)**

- **Effective seismic weight of the structure** $W = 5650.0$ kips
- **Seismic response coefficient** $C_s = 0.113$
- **Seismic base shear (Eq 12.8-1)** $V = C_s \times W = 636.9$ kips

**Vertical distribution of seismic forces** (Section 12.8.3)

- **Vertical distribution factor (Eq 12.8-12)** $C_{vx} = w_x \times h_x^k \times (w_i \times h_i^k)$
- **Lateral force induced at level $i$ (Eq 12.8-11)** $F_x = C_{vx} \times V$
Table 2-3. Vertical Force Distribution (X Direction)

<table>
<thead>
<tr>
<th>Level</th>
<th>Height from base to Level i (ft), ( h_x )</th>
<th>Portion of effective seismic weight assigned to Level i (kips), ( w_x )</th>
<th>Distribution exponent related to building period, ( k )</th>
<th>Vertical distribution factor, ( C_{vx} )</th>
<th>Lateral force induced at Level i (kips), ( F_x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>45.0</td>
<td>628.0</td>
<td>1.16</td>
<td>0.217</td>
<td>138.1</td>
</tr>
<tr>
<td>3rd</td>
<td>30.0</td>
<td>2511.0</td>
<td>1.16</td>
<td>0.541</td>
<td>344.7</td>
</tr>
<tr>
<td>2nd</td>
<td>15.0</td>
<td>2511.0</td>
<td>1.16</td>
<td>0.242</td>
<td>154.1</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>636.9</td>
</tr>
</tbody>
</table>

**Y-DIRECTION**

![Figure 2-7. Y-Direction Forces (E_y)](image)

**Seismic Forces (ASCE 7-10)**

**Site parameters**

- Site class: D
- Mapped acceleration parameters (Section 11.4.1)
  - at short period: \( S_s = 2.17 \)
  - at 1 sec period: \( S_t = 0.743 \)
- Site coefficient at short period (Table 11.4-1): \( F_s = 1.0 \)
- Site coefficient at 1 sec period (Table 11.4-2): \( F_v = 1.5 \)

**Spectral response acceleration parameters**

- at short period (Eq. 11.4-1): \( S_{MS} = F_s \times S_s = 2.17 \)
- at 1 sec period (Eq. 11.4-2): \( S_{M1} = F_v \times S_t = 1.11 \)
**Design of Composite RCS Special Moment Frames**

**Design spectral acceleration parameters (Section 11.4.4)**

at short period (Eq. 11.4-3) \( S_{DS} = \frac{2}{3} \times S_{MS} = 1.45 \)

at 1 sec period (Eq. 11.4-4) \( S_{D1} = \frac{2}{3} \times S_{M1} = 0.74 \)

**Seismic design category**

Risk category (Table 1.5-1) II

Seismic design category based on short period response acceleration (Table 11.6-1) D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2) D

**Calculated fundamental period**

Height above base to highest level of building \( h_n = 45 \) ft

*From Table 12.8-2:*

- Structure type: Steel moment frame
- Building period parameter \( C_t \) = 0.028
- Building period parameter \( x \) = 0.80
- Coefficient for upper limit on calculated period (Table 12.8-1) \( C_U = 1.4 \)
- Specified fundamental period \( T_{analysis} = 0.85 \) sec

**Approximate fundamental period (Eq 12.8-7)** \( T_a = C_t \times (h_n)^x \times \frac{1}{1\text{sec}} / (1\text{ft})^x = 0.59 \) sec

**Calculated fundamental period (Sect 12.8.2)** \( T = \min(T_{analysis}, C_U \times T_a) = 0.82 \) sec \( = T_{used} \)

**Long-period transition period** \( T_L = 8 \) sec

**Seismic response coefficient**

Seismic force-resisting system (Table 12.2-1, C, 8)

- Response modification factor (Table 12.2-1) \( R = 8 \)
- Seismic importance factor (Table 1.5-2) \( I_e = 1.0 \)

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) \( C_s_{calc} = \frac{S_{DS}}{R/I_e} = 0.181 \)

Maximum (Eq 12.8-3) \( C_s_{max} = \frac{S_{D1}}{(T \times (R/I_e))} = 0.113 \)

Minimum:

- Eq 12.8-5 \( C_s_{min1} = \max(0.044 \times S_{DS} \times I_e, 0.01) = 0.064 \)
- Eq 12.8-6 (where \( S_1 >= 0.6 \)) \( C_s_{min2} = \frac{0.5 \times S_1}{(R/I_e)} = 0.046 \)

Seismic response coefficient \( C_s = 0.113 \)
Seismic base shear (Sect 12.8.1)

Effective seismic weight of the structure \( W = 5650.0 \text{ kips} \)
Seismic response coefficient \( C_s = 0.113 \)
Seismic base shear (Eq 12.8-1) \( V = C_s \times W = 636.9 \text{ kips} \)

Vertical distribution of seismic forces (Section 12.8.3)

Vertical distribution factor (Eq 12.8-12) \( C_{vx} = \frac{w_x \times h_x^k}{\sum(w_i \times h_i^k)} \)
Lateral force induced at level \( i \) (Eq 12.8-11) \( F_x = C_{vx} \times V \)

Table 2-4. Vertical Force Distribution (Y Direction)

<table>
<thead>
<tr>
<th>Level</th>
<th>Height from base to Level ( i ) (ft), ( h_x )</th>
<th>Portion of effective seismic weight assigned to Level ( i ) (kips), ( w_x )</th>
<th>Distribution exponent related to building period, ( k )</th>
<th>Vertical distribution factor, ( C_{vx} )</th>
<th>Lateral force induced at Level ( i ) (kips), ( F_x )</th>
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</thead>
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<tr>
<td>Roof</td>
<td>45.0</td>
<td>628.0</td>
<td>1.16</td>
<td>0.217</td>
<td>138.1</td>
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<tr>
<td>3rd</td>
<td>30.0</td>
<td>2511.0</td>
<td>1.16</td>
<td>0.541</td>
<td>344.7</td>
</tr>
<tr>
<td>2nd</td>
<td>15.0</td>
<td>2511.0</td>
<td>1.16</td>
<td>0.242</td>
<td>154.1</td>
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<td>Total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>636.9</td>
</tr>
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</table>
2.3. Modeling and Analysis

23.1. 3D Model Conditions and Assignments

SAP2000 was used to generate the 3D model for this design example. All structural elements including steel beams, concrete columns and concrete slabs were modeled as elastic elements. The seismic base of the structure was defined at the first floor on grade. The foundation grade beams were modeled as part of the frame. The isolated footings were not modeled and any restraining effects of the soil on the footing or other soil-structure interaction were not considered.

Support Conditions

The columns were rigidly connected to the foundation grade beams, and pinned supports were assigned to the footings beneath the grade beams. In the model, this is achieved by assigning a fixed constraint to the base of the column. The foundation system is detailed to consist of isolated spread foundations interconnected with grade beams which provide the fixity.

Gravity System

Gravity beams were modeled as pin-ended elements with no lateral stiffness.

SFRS

The SFRS beams and columns were modeled with fixed end connections (using centerline dimensions).

Diaphragms

To simplify the analysis model in this example, “Diaphragm Constraint” by SAP2000 was assigned at each floor including the roof steel deck. Stress transfer by diaphragms was checked separately.

Mass/Weight

The mass and weight for concrete columns, concrete slabs and steel beams were self-included in the model and calculated by SAP2000. All other seismic mass and weight were manually assigned.

Stiffness

Finite joint size was neglected. Column & Beam stiffness is taken as \( I_{c0} = 0.7 \times I_{co} \) & \( I_{g} = 1.0 \times I_{go} \), respectively (See Appendix C).

- \( I_{co} \): Moment of inertia of gross concrete column section about centroidal axis, neglecting reinforcement.
- \( I_{go} \): Moment of inertia of bare steel beam.
Figure 2-8. SAP2000 Model Isometric View

Figure 2-9. Roof Framing Plan (SAP model)
DESIGN OF COMPOSITE RCS SPECIAL MOMENT FRAME

Figure 2-10. 2\textsuperscript{nd} & 3\textsuperscript{rd} Floor Framing Plan (SAP model)

Figure 2-11. 1\textsuperscript{st} Floor Framing Plan (SAP model)
Figure 2-12. Framing Elevation (1 & 4) (SAP model)

Figure 2-13. Framing Elevation (2 & 3) (SAP model)
Figure 2-14. Framing Elevation (A & D) (SAP model)

Figure 2-15. Framing Elevation (B & C) (SAP model)
2.3.2. Analysis Results

A summary of the results obtained from SAP 2000 model is included below. The following figures show the moment diagrams for the worst case beam and column for each load case. Table 2-5 and 2-6 show design demand for each load case.

G.L. 2

Figure 2-16. Member Location

Figure 2-17. Moment Diagram under Dead Load (Frame 2 & 3) (kip-ft)

Figure 2-18. Moment Diagram under Superimposed Dead Load (Frame 2 & 3) (kip-ft)
Figure 2-19. Moment Diagram under $L_r$ Load (Frame 2 & 3) (kip-ft)

Figure 2-20. Moment Diagram under Live Load (Frame 2 & 3) (kip-ft)

Figure 2-21. Moment Diagram under $E_x$ Load (Frame 2 & 3) (kip-ft)
### Table 2-5. Typical Design Force (CF5 on G.L.2) (kip-ft, kip)

<table>
<thead>
<tr>
<th>Member:W30x108</th>
<th>Direction:X</th>
<th>Location:2nd Floor</th>
<th>Glidline:2/B-C, B-end</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment (M)</td>
<td>M\text{Dead}</td>
<td>M\text{Super Dead}</td>
<td>M\text{Lr}</td>
</tr>
<tr>
<td>kip-ft</td>
<td>-134</td>
<td>-33</td>
<td>0</td>
</tr>
<tr>
<td>Shear (V)</td>
<td>V\text{Dead}</td>
<td>V\text{Super Dead}</td>
<td>V\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
<td>-14</td>
<td>-3</td>
<td>0</td>
</tr>
<tr>
<td>Axial (N)</td>
<td>N\text{Dead}</td>
<td>N\text{Super Dead}</td>
<td>N\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
<td>0</td>
<td>0</td>
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<th>Glidline:2/B-C, C-end</th>
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<tbody>
<tr>
<td>Moment (M)</td>
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<td>M\text{Super Dead}</td>
<td>M\text{Lr}</td>
</tr>
<tr>
<td>kip-ft</td>
<td>-135</td>
<td>-33</td>
<td>0</td>
</tr>
<tr>
<td>Shear (V)</td>
<td>V\text{Dead}</td>
<td>V\text{Super Dead}</td>
<td>V\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
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<td>3</td>
<td>0</td>
</tr>
<tr>
<td>Axial (N)</td>
<td>N\text{Dead}</td>
<td>N\text{Super Dead}</td>
<td>N\text{Lr}</td>
</tr>
<tr>
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<table>
<thead>
<tr>
<th>Member:GBx</th>
<th>Direction:X</th>
<th>Location:1st Floor</th>
<th>Glidline:2/B-C, B-end</th>
</tr>
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<tbody>
<tr>
<td>Moment (M)</td>
<td>M\text{Dead}</td>
<td>M\text{Super Dead}</td>
<td>M\text{Lr}</td>
</tr>
<tr>
<td>kip-ft</td>
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<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Shear (V)</td>
<td>V\text{Dead}</td>
<td>V\text{Super Dead}</td>
<td>V\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Axial (N)</td>
<td>N\text{Dead}</td>
<td>N\text{Super Dead}</td>
<td>N\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
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<th>Glidline:2/B-C, C-end</th>
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<tbody>
<tr>
<td>Moment (M)</td>
<td>M\text{Dead}</td>
<td>M\text{Super Dead}</td>
<td>M\text{Lr}</td>
</tr>
<tr>
<td>kip-ft</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Shear (V)</td>
<td>V\text{Dead}</td>
<td>V\text{Super Dead}</td>
<td>V\text{Lr}</td>
</tr>
<tr>
<td>kip</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Axial (N)</td>
<td>N\text{Dead}</td>
<td>N\text{Super Dead}</td>
<td>N\text{Lr}</td>
</tr>
<tr>
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Case 1 = (1.2+0.2x1.447)(D+SD)+E+L
Case 2 = (1.2+0.2x1.447)(D+SD)-E+L
Case 3 = (0.9-0.2x1.447)(D+SD)+E
Case 4 = (0.9-0.2x1.447)(D+SD)-E
Figure 2-22. Member Location

Figure 2-23. Moment Diagram under Dead Load (Frame A & D) (kip-ft)

Figure 2-24. Moment Diagram under Superimposed Dead Load
(Frame A & D) (kip-ft)
Figure 2-25. Moment Diagram under Lx Load (Frame A & D) (kip-ft)

Figure 2-26. Moment Diagram under Live Load (Frame A & D) (kip-ft)

Figure 2-27. Moment Diagram under Ey Load (Frame A & D) (kip-ft)
### Table 2-6. Typical Design Force (CF1 on G.L.A) (kip-ft, kip)

<table>
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<tr>
<th>Member: W30x108</th>
<th>Direction: Y</th>
<th>Location: 2nd Floor</th>
<th>Glidline: A/1-2, 1-end</th>
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<tr>
<td><strong>Moment (M)</strong></td>
<td><strong>M&lt;sub&gt;Dead&lt;/sub&gt;</strong></td>
<td><strong>M&lt;sub&gt;Super Dead&lt;/sub&gt;</strong></td>
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<td>Shear (V)</td>
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<td><strong>V&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<td><strong>N&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<td>0</td>
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<th>Member: W30x108</th>
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<tr>
<td><strong>Moment (M)</strong></td>
<td><strong>M&lt;sub&gt;Dead&lt;/sub&gt;</strong></td>
<td><strong>M&lt;sub&gt;Super Dead&lt;/sub&gt;</strong></td>
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<td>Shear (V)</td>
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<td><strong>V&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<td>kip</td>
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<td>35</td>
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<td>Axial (N)</td>
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<td><strong>N&lt;sub&gt;Super Dead&lt;/sub&gt;</strong></td>
<td><strong>N&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<tr>
<td>kip</td>
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<table>
<thead>
<tr>
<th>Member: GBy</th>
<th>Direction: Y</th>
<th>Location: 1st Floor</th>
<th>Glidline: A/1-2, 1-end</th>
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<tbody>
<tr>
<td><strong>Moment (M)</strong></td>
<td><strong>M&lt;sub&gt;Dead&lt;/sub&gt;</strong></td>
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<td><strong>N&lt;sub&gt;Super Dead&lt;/sub&gt;</strong></td>
<td><strong>N&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<td><strong>M&lt;sub&gt;L&lt;/sub&gt;</strong></td>
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<tr>
<td>kip-ft</td>
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<td>0</td>
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</table>

**Case 1** = (1.2 + 0.2 × 1.447)(D + SD) + E + L
**Case 2** = (1.2 + 0.2 × 1.447)(D + SD) - E + L
**Case 3** = (0.9 - 0.2 × 1.447)(D + SD) + E
**Case 4** = (0.9 - 0.2 × 1.447)(D + SD) - E
**Concrete Column Design Demand**

Orthogonal effects on the columns were considered. Biaxial bending was considered because orthogonal effects were not negligible. Per ASCE 7, Chapter 12, Section 5, 100 percent of the forces for one direction plus 30 percent of the force for perpendicular direction were used.

**C1 1st Floor G.L. Bx2**

<table>
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<tr>
<th>Member: C1</th>
<th>Position: Bottom</th>
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<th>Glline: 2/B</th>
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<td>EQX</td>
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<td>$V_{SD_3}$</td>
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<td>0</td>
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<tr>
<td>EQY</td>
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<td>$M_{D_3}$</td>
<td>$M_{SD_2}$</td>
<td>$M_{SD_3}$</td>
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<td>EQY</td>
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<tr>
<td>EQY</td>
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<td>$N_{D_3}$</td>
<td>$N_{SD_2}$</td>
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Case1 = (1.2+0.2x1.447)(D+SD)+E+L
Case2 = (1.2+0.2x1.447)(D+SD)-E+L
Case3 = (0.9-0.2x1.447)(D+SD)+E
Case4 = (0.9-0.2x1.447)(D+SD)-E
2.4. Seismic Design Requirements for Building Structure

2.4.1. Irregularities Check

The building did not have any irregularities. Building horizontal and vertical irregularities were evaluated per ASCE 7, Section 12.3.

2.4.2. Redundancy Factor ($\rho$)

The $\rho$ is 1.0 because this design example satisfies Condition (b) in ASCE 7 Section 12.3.4.2. For structures assigned to Seismic Design Category D, E, or F, $\rho$ shall equal 1.3 unless one of the condition in ASCE 7 12.3.4.2 is met, whereby $\rho$ is permitted to be taken as 1.0.

2.4.3. Drift Check

The building satisfied the design story drift requirements. The interstory drift ratio remained within the code limit (see Table 2-8). The building is stiffer in the Y-direction due to the smaller span length. The girders could possibly be optimized to achieve similar stiffness in both directions and have potential savings in steel tonnage. However, the savings in tonnage may be offset by the ease of joint construction using similar sections in both directions.

The design story drift ($\Delta$) is the story displacement at the center of mass of one floor level relative to the displacement of the center of mass at the floor level below. It is derived from a linear-elastic static analysis using the design seismic force by SAP2000. Calculated lateral displacements are multiplied by the deflection amplification factor ($C_d$) to include both elastic and estimated inelastic displacements resulting from design basis ground motion.

$$\delta x = \frac{C_d \times \delta xe}{l_e}, \delta y = \frac{C_d \times \delta ye}{l_e}$$

$$\Delta x = \delta x - \delta x_{below}, \Delta y = \delta y - \delta y_{below}$$

$C_d = 5.5, l_e = 1.0$

$h_{sx}$ and $h_{sy}$ are story heights in the X- and Y-directions, respectively.

$\delta xe$ and $\delta ye$ are lateral elastic displacements in X- and Y-directions, respectively.

$\delta x$ and $\delta y$ are lateral inelastic displacements in X- and Y-directions, respectively.

$\Delta x$ and $\Delta y$ are lateral interstory drifts in X- and Y-directions, respectively.
<table>
<thead>
<tr>
<th></th>
<th>$\Sigma h_x$(in)</th>
<th>$\delta x_e$(in)</th>
<th>$h_x$(in)</th>
<th>$\delta x$(in)</th>
<th>$\Delta x$(in)</th>
<th>$\Delta x/h_x$</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>X-dir</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Roof-3rd</td>
<td>540</td>
<td>2.0</td>
<td>180</td>
<td>10.9</td>
<td>3.7</td>
<td>0.021</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(13716)</td>
<td>(50.3)</td>
<td>(4572)</td>
<td>(276.5)</td>
<td>(94.3)</td>
<td></td>
<td>0.025</td>
</tr>
<tr>
<td>3rd-2nd</td>
<td>360</td>
<td>1.30</td>
<td>180</td>
<td>7.2</td>
<td>4.0</td>
<td>0.022</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(9144)</td>
<td>(33.1)</td>
<td>(4572)</td>
<td>(182.2)</td>
<td>(102.2)</td>
<td></td>
<td>0.025</td>
</tr>
<tr>
<td>2nd-1st</td>
<td>180</td>
<td>0.6</td>
<td>180</td>
<td>3.2</td>
<td>3.2</td>
<td>0.018</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(4572)</td>
<td>(14.5)</td>
<td>(4572)</td>
<td>(80.0)</td>
<td>(80.0)</td>
<td></td>
<td>0.025</td>
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<table>
<thead>
<tr>
<th></th>
<th>$\Sigma h_y$(in)</th>
<th>$\delta y_e$(in)</th>
<th>$h_y$(in)</th>
<th>$\delta y$(in)</th>
<th>$\Delta y$(in)</th>
<th>$\Delta y/h_y$</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Y-dir</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof-3rd</td>
<td>540</td>
<td>1.5</td>
<td>180</td>
<td>8.1</td>
<td>2.6</td>
<td>0.014</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(13716)</td>
<td>(37.2)</td>
<td>(4572)</td>
<td>(204.5)</td>
<td>(65.6)</td>
<td></td>
<td>0.025</td>
</tr>
<tr>
<td>3rd-2nd</td>
<td>360</td>
<td>1.0</td>
<td>180</td>
<td>5.5</td>
<td>3.0</td>
<td>0.017</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(9144)</td>
<td>(25.3)</td>
<td>(4572)</td>
<td>(138.9)</td>
<td>(76.5)</td>
<td></td>
<td>0.025</td>
</tr>
<tr>
<td>2nd-1st</td>
<td>180</td>
<td>0.4</td>
<td>180</td>
<td>2.5</td>
<td>2.5</td>
<td>0.014</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(4572)</td>
<td>(11.4)</td>
<td>(4572)</td>
<td>(62.4)</td>
<td>(62.4)</td>
<td></td>
<td>0.025</td>
</tr>
</tbody>
</table>
2.5. Member Design

2.5.1. Concrete Column Design

**Narrative**

The concrete columns were designed in accordance with ACI 318-11. Biaxial loading effects were considered in the concrete column designs.

**Typical Column (1\textsuperscript{st} Floor 2/B X-Direction)**

**Design information**

<table>
<thead>
<tr>
<th>Member</th>
<th>C1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>1\textsuperscript{st} Floor 2/B Bottom</td>
</tr>
<tr>
<td>Column Width</td>
<td>B=32 in</td>
</tr>
<tr>
<td>Column Depth</td>
<td>D=32 in</td>
</tr>
<tr>
<td>Specified concrete strength</td>
<td>f', = 6 ksi(normal weight concrete)</td>
</tr>
<tr>
<td>Vertical Rebar</td>
<td>(20) #11</td>
</tr>
<tr>
<td>Tie bar</td>
<td>(4)#5 @6&quot; OC EW</td>
</tr>
</tbody>
</table>

**Design force for column design**

| Table 2-9. Design Force of Column at the bottom of 1\textsuperscript{st} Floor on 2/B (kip-ft, kip) |
|---|---|---|---|---|---|---|---|
| Moment(M) | MX | MX | MX | MY | MX | MY | MX | MY |
| Ex+30%Ey | 172 | 718 | -174 | -692 | 173 | 708 | -173 | -702 |
| Ey+30%Ex | 564 | 228 | -567 | -202 | 565 | 218 | -566 | -212 |
| Shear(V) | VX | VX | VX | VX | VX | VX | VX | VX |
| Ex+30%Ey | 66 | 12 | -55 | -12 | 62 | 12 | -59 | -12 |
| Ey+30%Ex | 24 | 38 | -13 | -39 | 20 | 39 | -17 | -39 |
| Axial(N) | NX | NY | NX | NY | NX | NY | NX | NY |
| Ex+30%Ey | -938 | -938 | -998 | -998 | -208 | -208 | -268 | -268 |
| Ey+30%Ex | -941 | -941 | -995 | -995 | -210 | -210 | -265 | -265 |

The design of columns was performed by calculating a column axial force-moment capacity (P-Mx-My) interaction diagram. The spColumn software was used to calculate this in the design example. The resulting P-Mx-My diagram is shown in Table 2-10. As shown in the Table 2-11, the DCR for the column is less than 1.0. The column has adequate strength.

As shown later, the column strength is controlled by the strong-column weak-beam design check.
Table 2-10. Output from spColumn

<table>
<thead>
<tr>
<th>Load Case No</th>
<th>P_u (kip)</th>
<th>M_ux (k-ft)</th>
<th>M_uy (k-ft)</th>
<th>(\phi) M_{nx} (k-ft)</th>
<th>(\phi) M_{ny} (k-ft)</th>
<th>NA depth (in)</th>
<th>Dt depth (in)</th>
<th>(\varepsilon_1)</th>
<th>(\phi)</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>938.0</td>
<td>718.0</td>
<td>172.0</td>
<td>2003.3</td>
<td>479.9</td>
<td>2.79</td>
<td>17.7</td>
<td>36.0</td>
<td>0.0031</td>
<td>0.74</td>
</tr>
<tr>
<td>2</td>
<td>941.0</td>
<td>228.0</td>
<td>564.0</td>
<td>717.2</td>
<td>1774.2</td>
<td>3.15</td>
<td>19.8</td>
<td>38.4</td>
<td>0.0028</td>
<td>0.72</td>
</tr>
<tr>
<td>3</td>
<td>998.0</td>
<td>692.0</td>
<td>174.0</td>
<td>1965.5</td>
<td>494.2</td>
<td>2.84</td>
<td>18.4</td>
<td>36.3</td>
<td>0.0029</td>
<td>0.72</td>
</tr>
<tr>
<td>4</td>
<td>995.0</td>
<td>202.0</td>
<td>567.0</td>
<td>649.4</td>
<td>1822.9</td>
<td>3.22</td>
<td>19.8</td>
<td>37.9</td>
<td>0.0028</td>
<td>0.71</td>
</tr>
<tr>
<td>5</td>
<td>208.0</td>
<td>708.0</td>
<td>173.0</td>
<td>1923.8</td>
<td>470.1</td>
<td>2.72</td>
<td>11.4</td>
<td>34.7</td>
<td>0.0062</td>
<td>0.90</td>
</tr>
<tr>
<td>6</td>
<td>210.0</td>
<td>218.0</td>
<td>565.0</td>
<td>709.0</td>
<td>1837.6</td>
<td>3.25</td>
<td>14.0</td>
<td>37.6</td>
<td>0.0051</td>
<td>0.90</td>
</tr>
<tr>
<td>7</td>
<td>268.0</td>
<td>702.0</td>
<td>173.0</td>
<td>1965.4</td>
<td>484.4</td>
<td>2.80</td>
<td>11.9</td>
<td>35.0</td>
<td>0.0059</td>
<td>0.90</td>
</tr>
<tr>
<td>8</td>
<td>265.0</td>
<td>212.0</td>
<td>566.0</td>
<td>702.4</td>
<td>1875.2</td>
<td>3.31</td>
<td>14.3</td>
<td>37.6</td>
<td>0.0049</td>
<td>0.89</td>
</tr>
</tbody>
</table>

Case 1 = \((1.2+0.2\times1.447)(D+SD)+(Ex+0.3Ey)+L\)
Case 2 = \((1.2+0.2\times1.447)(D+SD)+(Ey+0.3Ex)+L\)
Case 3 = \((1.2+0.2\times1.447)(D+SD)-(Ex+0.3Ey)+L\)
Case 4 = \((1.2+0.2\times1.447)(D+SD)-(Ey+0.3Ex)+L\)
Case 5 = \((0.9-0.2\times1.447)(D+SD)+(Ex+0.3Ey)\)
Case 6 = \((0.9-0.2\times1.447)(D+SD)+(Ey+0.3Ex)\)
Case 7 = \((0.9-0.2\times1.447)(D+SD)-(Ex+0.3Ey)\)
Case 8 = \((0.9-0.2\times1.447)(D+SD)-(Ey+0.3Ex)\)
**Column Shear Check (ACI 318)**

**Design information**
- Factored Shear Force: \( V_u = 66 \text{ kip} \) from Table 2-9
- Modification Factor: \( \lambda = 1.0 \) (Nominal Weight Concrete)
- Specified Concrete Strength: \( f'_c = 6000 \text{ psi} \)
- Specified Yield Strength of Transverse Reinforcement: \( f_{yt} = 60,000 \text{ psi} \)
- Column Width: \( B = 32 \text{ in} \)
- Column Depth: \( D = 32 \text{ in} \)
- Column Core With: \( b_{c1} = 29.0 \text{ in} \) \( (B - 2 \times 1.5 \text{ in}) \)
- Column Core Depth: \( b_{c2} = 29.0 \text{ in} \) \( (D - 2 \times 1.5 \text{ in}) \)
- Gross Area of Column: \( A_g = B \times D = 1024 \text{ in}^2 \)
- Core Area of Column: \( A_{ch} = b_{c1} \times b_{c2} = 841 \text{ in}^2 \)

**Lo (21.6.4.1)**
- \( D = 32 \text{ in} \)
- \( 1/6 \times l_e = 25 \text{ in} \) Max. = 32 in
- 18 in

**So (21.6.4.3)**
- \( 1/4 \times B = 8 \text{ in} \)
- \( 6 \times 1.27 = 7.62 \text{ in} \) Min. = 4 in
- \( 4 + (14 - h_x)/3 = 4 \text{ in} \)

**Volume and strength check**

Cross-sectional Area of Transverse Reinforcement

**Reinforcement (21.6.4.4)**
- \( Ash_1 = 0.3 \times s \times b_{c1} \times f'_c / f_{yt} \times (A_g / A_{ch} - 1) \)
- \( = 0.757 \text{ in}^2 \) (Eq 21-4)
- \( Ash_2 = 0.09 \times s \times b_{c1} \times f'_c / f_{yt} \)
- \( = 1.044 \text{ in}^2 \) (Eq 21-5)
- USE : (4) #5 @ 4” O.C. \( (p_s = 0.010, Ash = 1.24 \text{ in}^2) \)

**Beyond Lo (21.6.4.5)**
- \( s_{max} = 6 \text{ in} \)
- \( d = 0.8 \times B = 25.6 \text{ in} \)
- \( V_c = 2 \times \lambda \times (\sqrt{(f'_c + 1 \text{ psi})} \times b_{c} \times d = 127 \text{ kip} \) (Eq 11-3)
- \( V_s = A_v \times f_{yt} \times d / s = 171 \text{ kip} \) (Eq 11-15)
- \( \phi V_n = 0.75 \times (V_c + V_s) = 224 \text{ kip} \)
- \( DCR = V_u / \phi V_n = 0.29 < 1.0 \text{ OK} \)
- USE : (4) #4 @ 6” O.C. \( (A_v = 0.8 \text{ in}^2) \)

Table 2-11 summarizes the concrete column design. As shown in the Table 2-11, the DCR for the columns is less than 1.0. The columns may appear to be over designed, however column capacity is governed by strong column weak beam requirement, which is presented by Section 2.5.3.
### Table 2-11. Summary of Column Design

<table>
<thead>
<tr>
<th>Member</th>
<th>Direction</th>
<th>Level</th>
<th>Location</th>
<th>Section</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3rd</td>
<td>2/B</td>
<td>Top 3&quot;x32&quot;(20)#10</td>
<td>0.13</td>
</tr>
<tr>
<td>C1</td>
<td>X</td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#10</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2nd</td>
<td>2/B</td>
<td>Top 3&quot;x32&quot;(20)#11</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#11</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1st</td>
<td>2/B</td>
<td>Top 3&quot;x32&quot;(20)#11</td>
<td>0.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#11</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3rd</td>
<td>2/A</td>
<td>Top 3&quot;x32&quot;(20)#10</td>
<td>0.15</td>
</tr>
<tr>
<td>C2</td>
<td>Y</td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#10</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2nd</td>
<td>2/A</td>
<td>Top 3&quot;x32&quot;(20)#11</td>
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<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#11</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1st</td>
<td>2/A</td>
<td>Top 3&quot;x32&quot;(20)#11</td>
<td>0.18</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(20)#11</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3rd</td>
<td>1/A</td>
<td>Top 3&quot;x32&quot;(16)#10</td>
<td>0.18</td>
</tr>
<tr>
<td>C3</td>
<td>Y</td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(16)#10</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2nd</td>
<td>1/A</td>
<td>Top 3&quot;x32&quot;(16)#11</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(16)#11</td>
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</tr>
<tr>
<td></td>
<td></td>
<td>1st</td>
<td>1/A</td>
<td>Top 3&quot;x32&quot;(16)#11</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 3&quot;x32&quot;(16)#11</td>
<td>0.27</td>
</tr>
</tbody>
</table>
### 2.5.2. Steel Beam Design

The design of the steel beams was performed in accordance with ASCE 7, AISC 360 and AISC 341. The size of the steel beam in a typical Composite RCS special moment frame tends to be governed by story drift requirements. The contribution of the composite slab may be considered to calculate the size of the beams, however in this design example, the steel beams were conservatively designed using the bare steel section properties only (following typical practice for steel moment frames). Although the steel beam may be designed ignoring the composite slab contributions, the effects of composite actions should be considered when performing strong column weak beams checks.

**Design Typical Beam (Bx 2F 2/B-C)**

#### Design information

<table>
<thead>
<tr>
<th>Member</th>
<th>Bx</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>2nd Floor, GL.2 between B &amp; C, B end</td>
</tr>
<tr>
<td>Beam Section</td>
<td>W30x108</td>
</tr>
<tr>
<td>Material</td>
<td>A992, Gr.50</td>
</tr>
</tbody>
</table>

#### Design force

<table>
<thead>
<tr>
<th>Moment (M)</th>
<th>kip-ft</th>
<th>M\text{Dead}</th>
<th>M\text{Super Dead}</th>
<th>M\text{Lr}</th>
<th>M\text{L}</th>
<th>M_{\text{case1}}</th>
<th>M_{\text{case2}}</th>
<th>M_{\text{case3}}</th>
<th>M_{\text{case4}}</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-135</td>
<td>-33</td>
<td>0</td>
<td>-219</td>
<td>-375</td>
<td>-845</td>
<td>-94</td>
<td>-478</td>
<td>273</td>
</tr>
<tr>
<td>Shear (V)</td>
<td>kip</td>
<td>V\text{Dead}</td>
<td>V\text{Super Dead}</td>
<td>V\text{Lr}</td>
<td>V\text{L}</td>
<td>V_{\text{case1}}</td>
<td>V_{\text{case2}}</td>
<td>V_{\text{case3}}</td>
<td>V_{\text{case4}}</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>3</td>
<td>0</td>
<td>22</td>
<td>15</td>
<td>63</td>
<td>33</td>
<td>26</td>
<td>-5</td>
</tr>
<tr>
<td>Axial (N)</td>
<td>kip</td>
<td>N\text{Dead}</td>
<td>N\text{Super Dead}</td>
<td>N\text{Lr}</td>
<td>N\text{L}</td>
<td>N_{\text{case1}}</td>
<td>N_{\text{case2}}</td>
<td>N_{\text{case3}}</td>
<td>N_{\text{case4}}</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

**Steel Member Design (AISC 360, LRFD)**

#### Section details

<table>
<thead>
<tr>
<th>Section type</th>
<th>W 30x108</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM steel designation</td>
<td>A992</td>
</tr>
<tr>
<td>Steel yield stress</td>
<td>$F_y = 50$ ksi</td>
</tr>
<tr>
<td>Steel tensile stress</td>
<td>$F_u = 65$ ksi</td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>$E = 29000$ ksi</td>
</tr>
</tbody>
</table>
**Resistance factors**

Resistance factor for tensile yielding \( \phi_{ty} = 0.90 \)

Resistance factor for tensile rupture \( \phi_{tr} = 0.75 \)

Resistance factor for compression \( \phi_c = 0.90 \)

Resistance factor for flexure \( \phi_b = 0.90 \)

Resistance factor for shear \( \phi_v = 1.00 \)

**Lateral bracing**

Length for major axis buckling \( L_x = 600 \text{ in} \)

Length for minor axis buckling \( L_y = 99.96 \text{ in} \)

Length for torsional buckling \( L_z = 99.96 \text{ in} \)

**Classification of Sections for Local Buckling - Section B4.1**

*Classification of flanges in flexure - Table B4.1b (case 10)*

Width to thickness ratio \( b_f / (2 \times t_f) = 6.91 \)

Limiting ratio for compact section \( \lambda_{pff} = 0.38 \times \sqrt{E / F_y} = 9.15 \)

Limiting ratio for non-compact section \( \lambda_{rff} = 1.0 \times \sqrt{E / F_y} = 24.08 \) Compact

*Classification of web in flexure - Table B4.1b (case 15)*

Width to thickness ratio \( (d - 2 \times k) / t_w = 49.50 \)

Limiting ratio for compact section \( \lambda_{pwf} = 3.76 \times \sqrt{E / F_y} = 90.55 \)

Limiting ratio for non-compact section \( \lambda_{rwf} = 5.70 \times \sqrt{E / F_y} = 137.27 \) Compact

*Section is compact in flexure*

**Design of members for shear parallel to y axis - Chapter G**

Required shear strength \( V_{ry} = 63.00 \text{ kips} \)

Web area \( A_w = d \times t_w = 16.24 \text{ in}^2 \)

Web plate buckling coefficient \( k_v = 5 \)

Web shear coefficient - eq G2-2 \( C_v = 1.00 \)

Nominal shear strength - eq G2-1 \( V_{nw} = 0.6 \times F_y \times A_w \times C_v = 487.23 \text{ kips} \)

Design shear strength \( V_{r} = \phi_v \times V_{nw} = 487.23 \text{ kips} \)

DCR = 0.13 < 1.0 OK

**PASS - Design shear strength exceeds required shear strength**

**Design of members for flexure in the major axis - Chapter F**

Required flexural strength \( M_r = 845 \text{ kips-ft} \)
**Yielding - Section F2.1**
Nominal flexural strength for yielding - Eq.F2-1 \( M_{nyld} = M_p = F_Y \times Z_x = 1441.67 \text{ kips ft} \)

**Lateral-torsional buckling - Section F2.2**
- Unbraced length: \( L_b = L_z = 99.96 \text{ in} \)
- Limiting unbraced length for yielding: Eq.F2-5 \( L_p = 1.76 \times r_y \times \sqrt{E / F_Y} = 91.13 \text{ in} \)
- Distance between flange centroids: \( h_o = d - t_f = 29.04 \text{ in} \)
  \( c = 1 \)
  \( r_{ts} = \sqrt{[(l_y \times C_w) / S_x]} = 2.67 \text{ in} \)
- Limiting unbraced length for inelastic LTB - Eq.F2-6
  \[ L_{r} = 1.95 \times r_{ts} \times E / (0.7 \times F_Y) \times \sqrt{(J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_Y / E)^2)}} = 264.23 \text{ in} \]
- Cross-section mono-symmetry parameter: \( R_m = 1.00 \)
- Lateral torsional buckling modification factor: \( C_b = 1.00 \)
- Nominal flexural strength for lateral torsional buckling - Eq.F2-2
  \[ M_{nltb} = C_b \times \left[ M_p - (M_p - 0.7 \times F_Y \times S_x) \times (L_b - L_o) / (L_r - L_o) \right] = 1412.61 \text{ kips ft} \]
- Nominal flexural strength: \( M_n = \min(M_{nyld}, M_{nltb}) = 1412.61 \text{ kips ft} \)
- Design flexural strength: \( M_c = \phi_b \times M_n = 1271.35 \text{ kips ft} \)
- DCR = 0.66 < 1.0 \ OK

**Stability Bracing of Beams (AISC 358)**
Steel beams were checked for stability and proportions per AISC 358, Section 5.3. As noted in Chapter 1, it is important to adequately brace the beam to inhibit lateral-torsional deformations and to promote a stable plastic hinge formation in the beams.

**Member Bx,W30x108(2, 3F)**

**Design information**
- Beam Depth: \( d = 29.8 \text{ in} \)
- Flange Width: \( b_f = 10.5 \text{ in} \)
- Flange Thickness: \( t_f = 0.76 \text{ in} \)
- Web Thickness: \( t_w = 0.545 \text{ in} \)
- Radius of Gyration: \( r_y = 2.15 \text{ in} \)
- Plastic Section Modulus: \( Z = 346 \text{ in}^3 \)
- Elastic Modulus: \( E = 29000 \text{ ksi} \)
- Yield Strength: \( F_Y = 50 \text{ ksi} \)
Check of the flange width-thickness ratio

\[ \frac{b}{t} = \frac{bf}{(2 \times tf)} = 6.91 \]  
(AISC 341, Table D 1.1)

\[ \frac{b}{t_{\text{max}}} = 0.3 \sqrt[3]{\frac{E}{Fy}} = 7.22 \]  
(AISC 341, Table D 1.1)

**OK,** \( \frac{b}{t} < \frac{b}{t_{\text{max}}} \)

Check of the web width-thickness ratio

\[ C_a = 0 \text{ for } C_a < 0.125, \text{ per footnote [d]} \]  
(AISC 341, Table D 1.1)

\[ \frac{h}{t_w} = \frac{d - 2 \times t_f}{t_w} = 51.9 \]  
(AISC 341, Table D 1.1)

\[ \frac{h}{t_w_{\text{max}}} = 2.45 \left( \sqrt[3]{\frac{E}{Fy}} \times (1 - 0.93 \times C_a) \right) = 59 \]  
(AISC 341, Table D 1.1)

**OK,** \( \frac{h}{t_w} < \frac{h}{t_w_{\text{max}}} \)

Maximum brace spacing, \( L_b \)

\[ L_b = 0.086 \times r_y \times \frac{E}{F_y} = 107 \text{ in, } 8.9 \text{ ft} \]  
(AISC 341, D1.2b)

Place minimum bracing 8’-4” on center

Brace strength design

\[ Ry = 1.1 \]

\[ Mr = Ry \times Z \times Fy = 19030 \text{ kip-in} \]  
(AISC 341, D1.2a)

\[ Cd = 1.0 \]

\[ h_o = d - t_f = 29.04 \text{ in} \]

\[ Prb = 0.02 \times Mr \times Cd / h_o = 13.1 \text{ kip} \]  
(AISC 341, D1.2a)

The length of the brace is assumed to be measured from the centerline of the W30x108 to the centerline of the adjacent gravity beam. Assuming 7.5 ft beam spacing, the length of the brace is space = 7.5 ft

\[ L = (\text{space}^2 + d^2)^{0.5} = 94.8 \text{ in} \]

Based on the Manual of Structural Steel Construction, a L3 1/2x3 1/2x3/8 as an eccentrically loaded single angle strength is

\[ \phi_c P_n = 18.5 \text{ kip (using 8 foot length)} \]  
(AISC Manual Table 4-12)

**OK,** \( \phi_c P_n > Prb \)
**Brace stiffness check**

Minimum stiffness per AISC 360 is
\[
\beta_{br} = 10 \times Mr \times Cd / (\phi \times L_b \times h_o) = 92.2 \text{ kips-in}
\]

The brace stiffness can be calculated as
\[
Ag = 2.50 \text{ in}^2 (L3 1/2x3 1/2x3/8)
\]
\[
\theta = \tan^{-1}(29.8 \text{ in} / 94.8 \text{ in}) = 17.5 \text{ degree}
\]

\[
K = \frac{Ag \cdot E}{L \cdot \cos^2 \theta} = 841 \text{ kip/in}
\]

OK, \( K > \beta_{br} \)

L3 1/2x3 1/2x3/8 brace provided at 8'-4" on center to brace the beam bottom flange to the top flange of an adjacent steel beam meet lateral bracing requirements.

![Figure 2-28. Brace Detail](image_url)

**Summary of Steel Beam**

Results of the steel beam flexural design are summarized in Table 2-13, where all of the DCRs are less than the limit of 1.0.

<table>
<thead>
<tr>
<th>Member</th>
<th>Floor</th>
<th>Location</th>
<th>Section</th>
<th>M(kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mu</td>
<td>( \phi ) Mn</td>
</tr>
<tr>
<td>Bx</td>
<td>RF</td>
<td>1/B-C</td>
<td>W21x62</td>
<td>177</td>
</tr>
<tr>
<td></td>
<td>2nd,3rd</td>
<td>2/B-C</td>
<td>W30x108</td>
<td>845</td>
</tr>
<tr>
<td>Gy</td>
<td>RF</td>
<td>A/1-2</td>
<td>W21x62</td>
<td>219</td>
</tr>
<tr>
<td></td>
<td>2nd,3rd</td>
<td>A/1-2</td>
<td>W30x108</td>
<td>915</td>
</tr>
<tr>
<td>B1</td>
<td>RF</td>
<td>-</td>
<td>W16x26</td>
<td>154</td>
</tr>
<tr>
<td></td>
<td>2nd,3rd</td>
<td>-</td>
<td>W24x68</td>
<td>759</td>
</tr>
<tr>
<td>G1</td>
<td>RF</td>
<td>-</td>
<td>W21x50</td>
<td>369</td>
</tr>
<tr>
<td></td>
<td>2nd,3rd</td>
<td>-</td>
<td>W30x108</td>
<td>1820</td>
</tr>
</tbody>
</table>

1: Composite Beam
2.5.3. Strong Column and Weak Beam (SCWB) Check

Strong column weak beam checks were performed in accordance with AISC 341. Columns should be designed to ensure that the plastic hinges are located in the beams. As discussed earlier, contribution from composite slab should be considered while performing SCWB checks. The strength ratio of column to beam of greater than 1.0 indicates a strong column & weak beam condition.

Typical Connection Check

The typical connection calculation at 2nd floor on GL 2 x B in the X-direction is shown below. A summary of each connection is included in Figures 2-29 & 2-30.

Expected Flexural Beam Strength (Mp.exp)
(Composite and Bare Steel Section, AISC 360 and Table 3-19)

Design information

Deck: W3 W/3.5"

Steel headed stud anchors: ¾" φ STUD @12” O.C.

Specified concrete strength: fc’ = 3 ksi (normal weight concrete)

Expected concrete strength: fc’exp = 1.3fc’ = 3.9 ksi (Blume Center Report #155 and LATBSDC 2014 Edition)

Modulus of elasticity of concrete: Ec,exp = 3560 ksi (for expected concrete strength)

Specified minimum tensile strength of a steel headed stud anchor: Fu = 75 ksi

Cross-sectional area of steel headed stud anchor: A_{sa} = 0.44 in²

Normal rib height: hr = 3 in

Average width of concrete rib or haunch: Wr = 6 in

Wr/hr = 2.0 > 1.5

Rg = 1.0 Rp = 0.75

Strength of Steel Headed Stud Anchors:

\[ Qn = \min(0.5 \times A_{sa} \times \sqrt{fc'_{exp} \times Ec_{exp}}, Rg \times Rp \times A_{sa} \times F_u) \]

= 24.8 kips (AISC 361 Eq 18-1)

Beam Span: Lx = 50 ft

Clear Distance to Next Web: ax = 30 ft / 4 = 7.5 ft

Spacing of Headed Stud Anchors: S = 12 in

Number of Studs: n = (Lx - 2 ft) / 2 / S = 24

Total Shear Strength of Stud: \( \Sigma Qn = n \times Qn = 595.2 \) kips

Depth of Slab: Ycon = 3 in + 3.5 in = 6.5 in
**Effective slab width**

\[
\frac{L_x}{8} = 6.25 \text{ ft} \\
ax / 2 \times 2 = 7.5 \text{ ft} \\
\Rightarrow b = \min \left( \frac{L_x}{8}, \frac{ax}{2}, b_{\text{col}} \right) = 32 \text{ in} \quad (\text{Blume Center Report #155})
\]

\[
b_{\text{col}} = 32 \text{ in (Column Width)}
\]

**Composite beam strength**

\[
a = \sum Q_n / (0.85 \times fc' \exp \times b) = 5.61 \text{ in}
\]

\[
Y_1 = \frac{d}{2} - \sum Q_n / (2 \times tw \times 1.1F_y) = 4.97 \text{ in}
\]

\[
Y_2 = Y_{\text{con}} - \frac{a}{2} = 3.70 \text{ in}
\]

\[
\phi M_p, \exp = 1878 \text{ kip-ft (Linear interpolation value of Table 3-19 in AISC Steel Construction Manual)}
\]

\[
1.1M_p, \exp' \text{pos} = \frac{\phi M_p, \exp}{0.9} \times 1.1 = 2295 \text{ kip-ft (Composite Beam Strength)}
\]

\[
1.1M_p, \exp' \text{neg} = 1.1 \times Z \times 1.1F_y / 12 = 1744 \text{ kip-ft (Bare Steel Beam Strength)}
\]

\[
1.1M_p, \exp' \text{pos} / 1.1M_p, \exp' \text{neg} = 1.32
\]

**Nominal Flexural Column Strength Above and Below Beam**

Nominal column flexural strength is calculated using the factored axial force associated with the earthquake load combination.

sp\text{Column} was used to calculate column strength under each axial force.

The minimum flexural strength shall be considered to calculate \( M_{p_{\text{cc}}} \).

### Table 2-14. Column Factored Strength at Each Axial Force

<table>
<thead>
<tr>
<th>Member: C1, Position: Top, Location: 2nd Floor, Gridline: 2/B, Direction: X</th>
<th>N</th>
<th>kip</th>
<th>M</th>
<th>kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>488</td>
<td>522</td>
<td>111</td>
<td>145</td>
</tr>
<tr>
<td>kip</td>
<td>2467</td>
<td>2483</td>
<td>2117</td>
<td>2146</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member: C1, Position: Bottom, Location: 2nd Floor, Gridline: 2/B, Direction: X</th>
<th>N</th>
<th>kip</th>
<th>M</th>
<th>kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>512</td>
<td>546</td>
<td>120</td>
<td>154</td>
</tr>
<tr>
<td>kip</td>
<td>2503</td>
<td>2530</td>
<td>2117</td>
<td>2146</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member: C1, Position: Top, Location: 1st Floor, Gridline: 2/B, Direction: X</th>
<th>N</th>
<th>kip</th>
<th>M</th>
<th>kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>914</td>
<td>974</td>
<td>198</td>
<td>258</td>
</tr>
<tr>
<td>kip</td>
<td>2802</td>
<td>2890</td>
<td>2211</td>
<td>2266</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member: C1, Position: Bottom, Location: 1st Floor, Gridline: 2/B, Direction: X</th>
<th>N</th>
<th>kip</th>
<th>M</th>
<th>kip-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>938</td>
<td>998</td>
<td>208</td>
<td>268</td>
</tr>
<tr>
<td>kip</td>
<td>2711</td>
<td>2722</td>
<td>2138</td>
<td>2184</td>
</tr>
</tbody>
</table>
Moment at Beam and Column Centerline ($\Sigma M^*_{p.exp}$, $\Sigma M^*_{pcc}$)

The expected flexural beam strength at the intersection of the beam and column centerlines are calculated using following formula:

**Definition of $\Sigma M^*_{p.exp}$**

$$\Sigma M^*_{p.exp} = \Sigma (1.1M_{p.exp} + M_{uv})$$

- $M_{p.exp}$ = Expected beam flexural strength, see previous section. (Kip-ft)
- $M_{uv}$ = Moment due to shear amplification from the location of the plastic hinge to the column centerline. (Kip-ft)
  
  $$M_{uv} = Emh \times Lp + Vg \times Lp$$

- $Emh$ = Amplified seismic load effect
  
  $$Emh = 2 \frac{1.1M_{p.exp}}{Lh}$$

- $Vg$ = Gravity shear at face of column (Kip)
- $Lh$ = Distance between plastic hinge location in the beam (ft)
- $Lp$ = Column Depth / 2 (ft)

**Note:** For C-SMF Frame, the plastic hinge is expected to form at the face of column.

**Calculation of $M_{p.exp}$**

See previous section for values.

$$\Sigma 1.1M_{p.exp} = 1744 + 2295 = 4039 \text{ kip-ft}$$

**Calculation of $M_{uv}$**

$$Emh = \frac{4039}{(50 - 32/12)} = 85.3 \text{ kip}$$

$$Vg = 31.4 \text{ kip} \quad \text{(See Table 2-12, 1.2D+0.5L)}$$

$$\Sigma M_{uv} = \left[ (85.3 + 31.4) \times \frac{32}{12} \right] \times 2 = 311 \text{ Kip-ft}$$

**Calculation of $\Sigma M^*_{p.exp}$**

$$\Sigma M^*_{p.exp} = 4039 + 311 = 4350 \text{ kip-ft}$$

The projections of the nominal flexural column strength at the intersection of the beam and column centerlines are calculated using following formula:

**Definition of $\Sigma M^*_{pcc}$**

$$\Sigma M^*_{pcc} = M_{pcc} \times ht / (ht - db/2)$$

- $M_{pcc}$ = Nominal column flexural strength, see previous section. (Kip-ft)
- $ht$ = Height of the column to its assumed points of inflection. (ft)
- $db$ = Beam depth (ft)

**Calculation of $\Sigma M^*_{pcc}$**

$$\Sigma M^*_{pcc} = 2211 \times \frac{15}{2} / \left( \frac{15}{2} - 29.8/12/2 \right) + 2141 \times \frac{15}{2} / \left( \frac{15}{2} - 29.8/12/2 \right) = 5215 \text{ Kip-ft}$$
Check of Flexural Strength Ratio of Column and Beam

SCWB calculation shall be evaluated using AISC 341 G3.4a.

\[ \Sigma M^p_{\text{exp}} = 4350 \text{kip-ft} \]
\[ \Sigma M^p_{\text{pcc}} = 5215 \text{kip-ft} \]
\[ \Sigma M^p_{\text{pcc}} / \Sigma M^p_{\text{exp}} = 5215 / 4350 = 1.20 \text{ OK} \]

Summary of Strong Column and Weak Beam Check

The summary of SCWB checks is included in Figure 2-29 and 2-30. As indicated, all of the ratios of column to beam strength are greater than 1.0.
2.5.4. Beam and Column Connection Design

The following calculations show the design of beam-to-column connections. Each connection was designed to achieve the connection strength governed by beam yielding. The strength of the joint is calculated per ASCE Pre-Standard. The connection strength is the summation of the inner panel strength and the outer panel strength. Inner panel strength is governed by the steel section. The Outer panel strength is governed by concrete strength and concrete column section. Provided that the required minimum reinforcing is provided, size and quantity of the rebar in the joint do not affect panel strength calculation. The main design parameters affecting the connection strength are (1) the geometry of the joint, (2) steel attachments to the beam (face bearing plates and band plates, web doubler plates, vertical joint reinforcement, etc.), and (3) concrete strength.

**Typical Connection Design (2nd Floor 2/B X-direction)**

**Design Information**

**Geometric information**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span</td>
<td>( L = 50 \text{ ft} )</td>
</tr>
<tr>
<td>Floor height</td>
<td>( H = 15 \text{ ft} )</td>
</tr>
</tbody>
</table>

**Beam section W30x108**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam height</td>
<td>( d = 29.8 \text{ in} )</td>
</tr>
<tr>
<td>Beam width</td>
<td>( \text{bf} = 10.5 \text{ in} )</td>
</tr>
<tr>
<td>Thickness of flange</td>
<td>( \text{tf} = 0.76 \text{ in} )</td>
</tr>
<tr>
<td>Thickness of web</td>
<td>( \text{tw} = 0.545 \text{ in} )</td>
</tr>
</tbody>
</table>

**Column section 32”x32”**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column width</td>
<td>( \text{b} = 32 \text{ in} )</td>
</tr>
<tr>
<td>Column height</td>
<td>( \text{h} = 32 \text{ in} )</td>
</tr>
<tr>
<td>Specified concrete strength</td>
<td>( f_c' = 6000 \text{ psi} )</td>
</tr>
</tbody>
</table>
**Calculation of Joint Strength**

**Inner panel strength**

\[ \phi V_{in} = \min(\phi_s x (V_{spn} + V_{icn}), \phi_b x (M_{vb} - V_b x h) / dj) \]  
(ASCE Eq 12)

- \( \phi_s = 0.85 \)
- \( \phi_b = 0.75 \)
- \( dj = d - tf = 29.04 \) in

\[ V_{spn} = 0.6 x F_{ysp} x t_{sp} x a_{sp} x h \]  
(ASCE Eq 13)

- \( F_{ysp} = 50 \) ksi
- \( t_{sp} = 0.545 \) in
- \( a_{sp} = 0.9 \)

\[ V_{spn} = 0.6 \times F_{ysp} \times t_{sp} \times a_{sp} \times h = 471 \text{ kip} \]

\[ V_{icn} = 1.7 x \alpha_c x (\text{sqrt}_{fc'si}) x b_i x h / 4448.3 \times 25.4^2 \]  
(ASCE Eq 14)

- \( \alpha_c = 1.0 \) for inner connection
- \( \text{sqrt}_{fc'si} = 6.48 \) ksi
- \( b_i = bf = 10.5 \) in

\[ V_{icn} = 1.7 \times \alpha_c \times (\text{sqrt}_{fc'si}) \times b_i \times h / 4448.3 \times 25.4^2 = 537 \text{ kip} \]

\[ dj = d - tf = 29.0 \text{ in} \]

\[ M_{vb} = C_{cn} x h x (1 - \beta_1 / 2) + h_{vr} x (T_{vrn} + C_{vrn}) \]  
(ASCE Eq 15)

- \( \alpha_{cn} = 2.5 \) for bearing regions with minimum re-bar ties
- \( \beta_1 = 0.75 \) per concrete strength
- \( C_{cn} = \alpha_{cn} \times f_c' \times bf \times (\beta_1 \times h / 2) = 1890 \text{ kip} \)  
(ASCE Eq 16)

- \( h_{vr} = h - 2 \times 3 \) in = 26 in
- \( T_{vrn} = 60 \) ksi \times 0 in\(^2\) = 0 kip
- \( C_{vrn} = 60 \) ksi \times 0 in\(^2\) = 0 kip

\[ M_{vb} = C_{cn} \times h \times (1 - \beta_1 / 2) + h_{vr} \times (T_{vrn} + C_{vrn}) = 3150 \text{ kips \_ft} \]

\[ V_b = (Mb1 + Mb2) / Lo \]

- \( Mb1 = 1744 \text{ kips \_ft} \) Expected bare beam flexural strength
- \( Mb2 = 1744 \text{ kips \_ft} \) Expected bare beam flexural strength

\[ Lo = L - h = 47.3 \text{ ft} \]

\[ V_b = (Mb1 + Mb2) / Lo = 73.7 \text{ kip} \]

**Inner panel strength**

\[ \phi V_{in} = \min(\phi_s x (V_{spn} + V_{icn}), \phi_b x (M_{vb} - V_b x h / 12) / (dj / 12)) = 857 \text{ kip} \]

**Outer panel strength**

\[ \phi V_{on} = \phi_s \times 1.25 \times \alpha_c \times (\text{sqrt}_{fc'si}) \times b_o \times h / 4448.3 \times 25.4^2 \]  
(ASCE Eq 18)

- \( \phi_s = 0.85 \)
- \( \alpha_c = 1.0 \) for inner connection
- \( y = bf = 10.5 \) in
\( \alpha_x = 0.5 \) for Steel band plate without ties in the bearing region above and below beam
\( X = h = 32 \text{ in} \) for Band Plates
\( b_o = y + \frac{2}{3} \times \alpha_x \times X - bf = 10.7 \text{ in} \)
\( \phi_b V_{on} = \phi_b \times 1.25 \times \alpha_c \times \sqrt{\text{fc'si}} \times b_o \times h / 4448.3 \times 25.4^2 = 341 \text{ kip} \)

**Joint strength**

\[ k = 1.00 \]
\[ \phi V_n = k \times (\phi V_{in} + \phi_b V_{on}) = 1197 \text{ kip} \quad (\text{ASCE Eq 10}) \]

**Calculation of joint force**

\[ \Sigma Mb = (Mb1 + Mb2) = 3488 \text{ kips ft} \]
\[ V_c = (Mb1 + V_b \times (h / 2) + Mb2 + V_b \times (h / 2)) / H = 246 \text{ kip} \]
\[ V_j = \Sigma Mb / dj - V_c = 1196 \text{ kip} \quad (\text{ASCE Eq 11}) \]

**Capacity check**

\[ \text{DCR} = V_j / \phi V_n = 0.998 \]
\[ V_j < \phi V_n \text{ OK} \]

**Summary of Connection Design**

Table 2-15 shows the summary of connection design. As shown in the Table 2-15, every DCR for connection design is less than the limit of 1.0.

<table>
<thead>
<tr>
<th>Level</th>
<th>Location</th>
<th>Direction</th>
<th>( V_{on}(\text{kip}) )</th>
<th>( V_{in}(\text{kip}) )</th>
<th>( \alpha_x )</th>
<th>( \phi V_{in}(\text{kip}) )</th>
<th>( \phi V_{on}(\text{kip}) )</th>
<th>( \phi V_{n}(\text{kip}) )</th>
<th>( V_j(\text{kip}) )</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>GL2 x GL B</td>
<td>X</td>
<td>346</td>
<td>421</td>
<td>0.5</td>
<td>401</td>
<td>652</td>
<td>341</td>
<td>993</td>
<td>804</td>
</tr>
<tr>
<td>2nd</td>
<td>GL2 x GL B</td>
<td>X</td>
<td>471</td>
<td>537</td>
<td>0.5</td>
<td>401</td>
<td>857</td>
<td>341</td>
<td>1197</td>
<td>1196</td>
</tr>
<tr>
<td>RF</td>
<td>GL2 x GL A</td>
<td>Y</td>
<td>346</td>
<td>421</td>
<td>0.5</td>
<td>401</td>
<td>652</td>
<td>341</td>
<td>993</td>
<td>802</td>
</tr>
<tr>
<td>2nd</td>
<td>GL2 x GL A</td>
<td>Y</td>
<td>471</td>
<td>537</td>
<td>0.5</td>
<td>401</td>
<td>857</td>
<td>341</td>
<td>1197</td>
<td>1186</td>
</tr>
</tbody>
</table>

**2.5.5. Connection Detail Design**

**Narrative**

The following calculations show the detailed design of beam & column interior and exterior connections. Each connection detail shall be designed to achieve the inner panel and outer panel strength of the connection. The connection was designed using the ASCE Pre-Standard.
**Design Information**

**Design Stress (Requirement)**
- Outer Panel Strength \( V_{on} = 401 \) kip from Table 2-15
- Inner Panel Strength \( V_{fcn} = 537 \) kip from Table 2-15

**Configuration**
- Concrete Column Size \( b = 32 \) in
- Concrete Column Size \( h = 32 \) in
- Band Plate Height \( d_{bp} = 8 \) in

**Tie Bar**
- \((4) \#5 @4'' OC EW\)
- Area of Tie Bar \( A_{tie} = 0.31 \) in\(^2\)
- Tie Bar Legs \( \ell = 4 \)
- Tie Bar Space \( s_{h} = 4 \) in
**Band Plate**

Yield Strength of Steel Band Plate \( F_{ybp} = 50 \text{ ksi} \)

Tensile Strength of Steel Band Plate \( F_{ubp} = 65 \text{ ksi} \)

Band Plate Thickness \( t_{bp} = 0.875 \text{ in} \)

**Face Bearing Plate**

Yield Strength of Face Bearing Plate \( F_{yp} = 50 \text{ ksi} \)

Tensile Strength of Face Bearing Plate \( F_{up} = 65 \text{ ksi} \)

Face Bearing Plate Thickness \( t_{p} = 0.875 \text{ in} \)

**Longitudinal Column Re-Bars**

\( (20)\#11 \)

Number of Vertical Bars \( n_{v} = 20 \)

Vertical Bar Diameter \( d_{b} = 1.41 \text{ in} \)

Yield Strength of Longitudinal Column Bars \( F_{yr} = 60 \text{ ksi (F}_{yr_{-si}} = 420 \text{ MPa} \)

**Steel Beam**

W30x108

Depth of Beam \( d = 29.8 \text{ in} \)

Width of Beam \( b_{f} = 10.5 \text{ in} \)

Thickness of Web \( t_{sp} = 0.55 \text{ in} \)

Thickness of Flange \( t_{f} = 0.76 \text{ in} \)

Effective Joint Depth \( d_{j} = d - t_{f} = 29.04 \text{ in} \)

Yield Strength of Web \( F_{ysp} = 50 \text{ ksi} \)

Yield Strength of Flange \( F_{yf} = 50 \text{ ksi} \)

The Flange Width of Steel Band Plate \( b'_{p} = 10.5 \text{ in} \)

**Check of Connection Detail**

**Tie Bar Check**

1) Maximum Spacing Between Horizontal Ties

\( \min(0.25 \times d_{j}, 0.25 \times h) = 7.26 \text{ in} \)

OK, 4” provided
2) Volumetric Ratio of Horizontal Rebar
\[ \rho_{s, \text{require}} = 0.01, \ V_{s, \text{require}} = \rho_{s, \text{require}} \times (dj \times h \times b) = 297.4 \text{ in}^3 \]
\[ V_{s, \text{design}} = \text{legs} \times As_{-\text{tie}} \times \{(b - 2 \times 2"") + (h - 2 \times 2"")\} \times dj / s_h = 504.1 \text{ in}^3 \]
\[ \text{OK, } V_{s, \text{design}} > V_{s, \text{require}} \]

**Concrete Confinement Check**

1) Band Plate Extension
\[ 0.25 \times dj = 7.26 \text{ in} \]
\[ \text{OK, } d_{bp} = 8 \text{ in} > 7.26 \text{ in} \]

2) Minimum Volume
\[ V_{bp, \text{min}} = 0.03 \times dj \times h \times b = 892 \text{ in}^3 \]
\[ V_{bp} = t_{bp} \times d_{bp} \times 2 (b + h) = 896 \text{ in}^3 \]
\[ \text{OK, } V_{bp} > V_{bp, \text{min}} \]

3) Band Plate Area
\[ A_{bp, \text{min}} = V_{on} / F_{ybp} = 8.0 \text{ in}^2 \]
\[ A_{bp} = t_{bp} \times d_{bp} \times 2 = 14.0 \text{ in}^2 \]
\[ \text{OK, } A_{bp} > A_{bp, \text{min}} \]

**Longitudinal Column Re-Bars Check**

\[ d_{b, \text{max}} = \frac{dj (420)}{20F_{yr}} \]  
(ASCE Eq 20)
\[ dj' = 1.25 \times dj = 36.3 \text{ in} \text{ (Steel Band Plate Are Used)} \]
\[ d_{b, \text{max}} = dj' \times 420 \text{ MPa} / (20 \times F_{yr, si}) = 1.82 \text{ in} \]
\[ \text{OK, } d_b = 1.41 \text{ in} < d_{b, \text{max}} \]

**Steel Beam Flanges Check**

\[ t_{f, \text{min}} = 0.30 \sqrt{\frac{b_f \times t_{sp} \times d \times F_{ysp}}{h \times F_{yf}}} \]  
(ASCE Eq 21)
\[ t_{f, \text{min}} = 0.30 \sqrt{((b \times t_{sp} \times d \times F_{ysp})/(h \times F_{yf}))} = 0.70 \text{ in} \]
\[ \text{OK, } t_{f, \text{provided}} = 0.76 \text{ in} > t_{f, \text{min}} \]

**Minimum Thickness of Band Plate Check**

\[ t_{bp, \text{min}} = 0.12 \sqrt{\frac{V_{on} \times b_p}{d_{bp} \times F_{ybp}}} \]  
(ASCE Eq 25)
\[ \text{Depth of Steel Band Plate } d_{bp} = 8.0 \text{ in} \]
\[ t_{bp, \text{min}} = 0.12 \sqrt{((V_{on} \times b_p')/(d_{bp} \times F_{ybp}))} = 0.39 \text{ in} \]
\[ t_{bp,\text{min}} = \frac{\sqrt{3}}{2b_p F_{ubp}} \cdot V_{on} \quad \text{(ASCE Eq 26)} \]

\[ t_{bp,\text{min}} = \left(\frac{\sqrt{3}}{2}\right) \times \left(2 \times b_p \times F_{ubp}\right) \times V_{on} = 0.51 \text{ in} \]

\text{OK, } t_{bp,\text{provided}} = 0.875 \text{ in} > t_{bp,\text{min}}

\text{Face Bearing Plates Check}

\[ t_{p,\text{min}} = \frac{\sqrt{3}}{2 \times b_f F_{up}} \times V_{icn} \quad \text{(ASCE Eq 22)} \]

\[ t_{p,\text{min}} = \left(\frac{\sqrt{3}}{2}\right) \times (2 \times b_f \times F_{up}) \times V_{icn} = 0.68 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \times \left(\frac{V_{icn} b_f}{F_{up} d_w}\right) \quad \text{(ASCE Eq 23)} \]

\[ d_w = (d - 2 \times t_f) = 28.28 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \times \left(\frac{V_{icn} b_f}{F_{up} d_w}\right) = 0.40 \text{ in} \]

\[ t_{p,\text{min}} = \frac{b_f}{22} \quad \text{(ASCE Eq 24)} \]

\[ t_{p,\text{min}} = b_f / 22 = 0.48 \text{ in} \]

\text{OK, } t_{p,\text{provided}} = 0.875 \text{ in} > t_{p,\text{min}}
**Design Information**

**Design Stress (Requirement)**

- Outer Panel Strength: \( V_{on} = 401 \text{ kip from Table 2-15} \)
- Inner Panel Strength: \( V_{icn} = 421 \text{ kip from Table 2-15} \)

**Configuration**

- Concrete Column Size: \( b = 32 \text{ in} \)
- Concrete Column Size: \( h = 32 \text{ in} \)
- Band Plate Height: \( d_{bp} = 6.5 \text{ in} \)

**Tie Bar**

- (4) #5 @4” OC EW
- Area of Tie Bar: \( A_{\text{tie}} = 0.31 \text{ in}^2 \)
- Tie Bar Legs: \( \text{legs} = 4 \)
- Tie Bar Space: \( s_h = 4 \text{ in} \)
**Band Plate**

- Yield Strength of Steel Band Plate \( F_{ybp} = 50 \text{ ksi} \)
- Tensile Strength of Steel Band Plate \( F_{ubp} = 65 \text{ ksi} \)
- Band Plate Thickness \( t_{bp} = 0.875 \text{ in} \)

**Face Bearing Plate**

- Yield Strength of Face Bearing Plate \( F_{yp} = 50 \text{ ksi} \)
- Tensile Strength of Face Bearing Plate \( F_{up} = 65 \text{ ksi} \)
- Face Bearing Plate Thickness \( t_{p} = 0.875 \text{ in} \)

**Longitudinal Column Re-Bars**

- (20)#10
- Number of Vertical Bar \( n_v = 20 \)
- Vertical Bar Diameter \( d_b = 1.27 \text{ in} \)
- Yield Strength of Longitudinal Column Bars \( F_{yr} = 60 \text{ ksi} \) (\( F_{yr,si} = 420 \text{ MPa} \))

**Steel Beam**

- W21x62
- Depth of Beam \( d = 21 \text{ in} \)
- Width of Beam \( b_f = 8.24 \text{ in} \)
- Thickness of Web \( t_{sp} = 0.4 \text{ in} \)
- Thickness of Flange \( t_f = 0.615 \text{ in} \)
- Effective Joint Depth \( d_j = d - t_f = 20.39 \text{ in} \)
- Yield Strength of Web \( F_{ysp} = 50 \text{ ksi} \)
- Yield Strength of Flange \( F_{yf} = 50 \text{ ksi} \)
- The Flange Width of Steel Band Plate \( b'_p = 8.24 \text{ in} \)

**Check of Connection Detail**

**Tie Bar Check**

1) Maximum Spacing Between Horizontal Ties

\[
\min(0.25 \times d_j, 0.25 \times h) = 5.10 \text{ in}
\]

**OK, 4” provided**
2) Volumetric Ratio of Horizontal Rebar

\[ \rho_{s,require} = 0.01, \ V_{s,require} = \rho_{s,require} \times (dj \times h \times b) = 208.8 \text{ in}^3 \]

\[ V_{s,design} = \text{legs} \times As_{tie} \times \{(b - 2 \times 2") + (h - 2 \times 2")\} \times dj / s_h = 354.0 \text{ in}^3 \]

OK, \( V_{s,design} > V_{s,require} \)

**Concrete Confinement Check**

1) Band Plate Extension

\[ 0.25 \times dj = 5.10 \text{ in} \]

OK, \( d_{bp} = 6.5 \text{ in} > 5.10 \text{ in} \)

2) Minimum Volume

\[ V_{bp,min} = 0.03 \times dj \times h \times b = 626 \text{ in}^3 \]

\[ V_{bp} = t_{bp} \times d_{bp} \times 2( b + h ) = 728 \text{ in}^3 \]

OK, \( V_{bp} > V_{bp,min} \)

3) Band Plate Area

\[ A_{bp,min} = V_{on} / F_{ybp} = 8.0 \text{ in}^2 \] \hspace{1cm} (ASCE Eq 19)

\[ A_{bp} = t_{bp} \times d_{bp} \times 2 = 11.4 \text{ in}^2 \]

OK, \( A_{bp} > A_{bp,min} \)

**Longitudinal Column Re-Bars Check**

\[ d_{b,\text{max}} = \frac{dj(420)}{20F_{yr}} \] \hspace{1cm} (ASCE Eq 20)

\[ dj' = 1.25 \times dj = 25.5 \text{ in} \] (Steel Band Plate Are Used)

\[ d_{b,\text{max}} = dj' \times 420 \text{ MPa} / (20 \times F_{yr,SI}) = 1.275 \text{ in} \]

OK, \( d_b = 1.27 \text{ in} < d_{b,\text{max}} \)

**Steel Beam Flanges Check**

\[ t_{f,min} = 0.30 \sqrt{\frac{b_f t_{sp} d F_{ysp}}{h F_{yf}}} \] \hspace{1cm} (ASCE Eq 21)

\[ t_{f,min} = 0.30 \times \sqrt{(b \times t_{sp} \times d \times F_{ysp})/(h \times F_{yf})} = 0.44 \text{ in} \]

OK, \( t_{f,\text{provided}} = 0.615 \text{ in} > t_{f,min} \)

**Minimum Thickness of Band Plate Check**

\[ t_{bp,min} = 0.12 \sqrt{\frac{V_{on} b_p}{d_{bp} F_{ybp}}} \] \hspace{1cm} (ASCE Eq 25)

Depth of Steel Band Plate \( d_{bp} = 6.5 \text{ in} \)

\[ t_{bp,min} = 0.12 \times \sqrt{(V_{on} \times b_p')/(d_{bp} \times F_{ybp})} = 0.43 \text{ in} \]
\[ t_{bp,\text{min}} = \frac{\sqrt{3}}{2b_p F_{ubp}} \cdot V_{on} \]  
\text{(ASCE Eq 26)}

\[ t_{bp,\text{min}} = \left( \frac{\sqrt{3}}{2 b_p F_{ubp}} \right) \cdot V_{on} = 0.65 \text{ in} \]

**OK, \( t_{bp,\text{provided}} = 0.875 \text{ in} > t_{bp,\text{min}} \)**

### Face Bearing Plates Check

\[ t_{p,\text{min}} = \frac{\sqrt{3}}{2 b_f F_{up}} \cdot V_{ic} \]  
\text{(ASCE Eq 22)}

\[ t_{p,\text{min}} = \left( \frac{\sqrt{3}}{2 b_f F_{up}} \right) \cdot V_{ic} = 0.68 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \sqrt{\frac{V_{ic} \cdot b_f}{F_{yp} \cdot d_w}} \]  
\text{(ASCE Eq 23)}

\[ d_w = (d - 2t_r) = 19.77 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \times \sqrt{\frac{V_{ic} \times b_f}{F_{yp} \times d_w}} = 0.37 \text{ in} \]

\[ t_{p,\text{min}} = \frac{b_f}{22} \]  
\text{(ASCE Eq 24)}

\[ t_{p,\text{min}} = b_t = b_t / 22 = 0.37 \text{ in} \]

**OK, \( t_{p,\text{provided}} = 0.875 \text{ in} > t_{p,\text{min}} \)**

### 2.6. Typical Detail and Construction Procedure

This section discusses the typical construction procedures and details for a Composite RCS systems. While the discussion is not comprehensive, because it does not consider all possible options, it is intended to encourage the engineer to consider the practical construction aspects associated with Composite RCS design. RCS frames consist of three main components, the concrete columns, steel beams, and the beam column connections. It is important to understand the construction practices and the construction technology where the project is being constructed in order to design and detail the system. For the purposes of this document, we have chosen to discuss cast-in-place (CIP) and precast concrete (PC) construction procedures. Both CIP and PC systems are acceptable per AISC and are designed similarly; however the detailing and construction sequence of the systems is quite different. The size of the project, availability of material, construction practices and availability of labor are some of the factors that should be considered when selecting the concrete system to be used. To the extent possible, it is advisable to involve the contractor early on the design and obtain feedback. Alternatively, the system can be designed to allow for both options (CIP and PC), thus allowing the contractor to choose either option.

#### 2.6.1. Cast in Place (CIP) Procedure

The sketches and details in this section represent one possible construction sequence for cast in place concrete construction. The sequence presented may be competitive with pre-cast concrete procedure given adequate construction management is performed. In the CIP procedure, the concrete and steel subcontractors have to work in tandem, and coordination between rebar and steel subcontractor is important. This procedure may be well suited when the building has a large foot print and work can be phased in segments. If the contractor chooses, a steel erection column may be included inside the column rebar cage, helping in erecting all steel at once. Once the steel is erected, the concrete may be cast all at once, or in phases as desired.
Splice location and detailing should be coordinated between the contractor and design engineer to facilitate handling, transportation and erection of columns while also ensuring reliable seismic performance. It is expected that the joint will be checked first, followed by the installation of the beams. The location of splice in beam should be selected such that the joint connection detail is construction friendly. Locating the splice at ¼ the span will help minimize the beam lower moment and shear demands, resulting in fewer bolts and thinner splice plates.

Figure 2-35. Cast in Place Procedure (1)
Figure 2-36. Cast in Place Procedure (2)
Typical Detail for Cast in Place Procedure

2nd & 3rd Floor

Figure 2-37. Beam & Column Connection Detail at the 2nd & 3rd Floors (Cast in Place)
Figure 2-38. Beam & Column Connection Detail at Roof Floor (Cast in Place)
Figure 2-39. Reinforcement End Plate Detail

Figure 2-40. Tie Detail at Joint (Cast in Place)
Sample Detail of Cover Plate

2nd & 3rd Floor

Figure 2-41. Beam & Column Connection Detail at the 2nd & 3rd Floors (Cast in Place) - Cover Plate Version -
Figure 2-42. Beam & Column Connection Detail at Roof Floor (Cast in Place) - Cover Plate Version -
2.6.2. Pre-Cast Procedure

The sketches and details in this section represent one possible construction sequence for pre-cast concrete construction. This procedure, when implemented effectively, can have a reduced construction schedule compared to a cast in place procedure. The precast construction procedure is similar to conventional steel construction as the precast concrete columns, beams and joints are prefabricated and erected on site. Splice location and detailing should be coordinated between the contractor and design engineer to facilitate handling, transportation and erection of columns while also ensuring reliable seismic performance.

Figure 2-43. Pre-Cast Procedure (1)
Figure 2-44. Pre-Cast Procedure (2)
Figure 2-45. Configuration of Pre-Cast Pieces
Typical Detail for Pre-cast Procedure. (Cover plate option is also applicable for pre-cast procedure).

2nd & 3rd Floor

Figure 2-46. Beam & Column Connection Detail at the 2nd & 3rd Floors (Pre-Cast)
Figure 2-47. Beam & Column Connection Detail at Roof Floor (Pre-Cast)
2.6.3. Sample Detail for Two Way System

2\textsuperscript{nd} & 3\textsuperscript{rd} Floor

Figure 2-48. Sample Details of Beam & Column Connection at the 2\textsuperscript{nd} & 3\textsuperscript{rd} Floors (Two Way System)
3. DESIGN EXAMPLE OF COMPOSITE RCS SPECIAL MOMENT FRAME WITH VISCOUS DAMPER

3.1. General Information

3.1.1. Narrative

Seismic design with viscous dampers has been used in the United States since the early 1990s. Viscous dampers can provide large equivalent damping and improve the performance of buildings such that the building can remain essentially elastic at the design basis earthquake (DBE) level and possibly higher levels. The use of viscous dampers along with a composite special moment frame is considered in this chapter. The building used in the example in Chapter 2 is considered.

The seismic force resisting system consists of Composite Special Moment Resisting Frames with concrete columns and steel beams. Supplemental damping is introduced in the building in the form of viscous dampers which are laid out in a balanced and efficient manner to avoid torsional irregularity and to optimize building response. The building is evaluated at the DBE level ground motion intensity. Nonlinear response history analysis (NLRHA) procedure per ASCE 7 is used for the analysis of the composite RCS frame with damper system. Figures 3-1 through 3-4 illustrate the geometry of the building and Table 3-1 summarizes the structural framing members.

An essentially elastic response (EER) is achieved using smaller steel beam and concrete column sizes than in the conventional Composite Special Moment Frame design (in Chapter 2). EER is achieved by ensuring that member force demands do not exceed the capacity of the selected members, except for only a few cases where some limited inelastic/nonlinear behavior is accepted. As outlined in this chapter, EER is achieved with a response reduction factor of R=1.0, illustrating that larger member sizes are not required to remain essentially elastic and that supplemental systems such as viscous dampers placed strategically can improve the building performance. The damped building has a reduced drift of 1.1% (compared to 2.2% in Chapter 2), 13% reduction in steel beam weight and 7% reduction in column sizes.

This following design steps are included in the example. For design information not contained in this chapter, see Chapter 2. Only additions and modification to Chapter 2 are included in Chapter 3.

1) Load Criteria & Load Combination
2) Acceleration Records
3) Modeling and Analysis
4) Seismic Design Requirement Check
5) Member & Connection Design
6) Typical Details
3.1.2. Geometry Definition

Figure 3-1. Typical Floor Framing Plan

Figure 3-2. Foundation Floor Framing Plan
Figure 3-3. Damper Frame Elevation of Frame (1, 4)

Figure 3-4. Damper Frame Elevation of Frame (A, D)
**Structural Member**

Table 3-1 summarizes the structural member sizes for the frame with viscous dampers.

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Size</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bx,Gy</td>
<td>RF</td>
<td>W21x62</td>
<td>Moment Frame Beam</td>
</tr>
<tr>
<td></td>
<td>3rd</td>
<td>W27x94</td>
<td>Moment Frame Beam</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>W27x94</td>
<td>Moment Frame Beam</td>
</tr>
</tbody>
</table>

**Table 3-1. Structural Member**

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Size (in x in)</th>
<th>Reinforcing</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>GBx</td>
<td>1st</td>
<td>30 x 40 (762 x 1016)</td>
<td>(6) #10 T&amp;B</td>
<td>#4 ties at 8” OC</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Member</th>
<th>Level</th>
<th>Size (in x in)</th>
<th>Reinforcing</th>
<th>Ties</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>3rd</td>
<td>30 x 30 (762 x 762)</td>
<td>(20) #10(V)</td>
<td>#4 ties at 6” o.c. (4 legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>30 x 30 (762 x 762)</td>
<td>(20) #11(V)</td>
<td>#5 ties at 4” o.c. (4 legs each way)</td>
</tr>
<tr>
<td>C2</td>
<td>3rd</td>
<td>30 x 30 (762 x 762)</td>
<td>(20) #10(V)</td>
<td>#4 ties at 6” o.c. (4 legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>30 x 30 (762 x 762)</td>
<td>(20) #11(V)</td>
<td>#5 ties at 4” o.c. (4 legs each way)</td>
</tr>
<tr>
<td>C3</td>
<td>3rd</td>
<td>30 x 30 (762 x 762)</td>
<td>(16) #10(V)</td>
<td>#4 ties at 6” o.c. (4 legs each way)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>30 x 30 (762 x 762)</td>
<td>(16) #11(V)</td>
<td>#5 ties at 4” o.c. (4 legs each way)</td>
</tr>
</tbody>
</table>

1: Lo Zone (ACI 318 21.6.4.3)

<table>
<thead>
<tr>
<th>Direction</th>
<th>Level</th>
<th>Viscous Damper</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>3rd</td>
<td>(4) Maximum Force 100kips (c=25kips/s/in)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>(4) Maximum Force 300kips (c=25kips/s/in)</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>(4) Maximum Force 300kips (c=25kips/s/in)</td>
</tr>
<tr>
<td>Y</td>
<td>3rd</td>
<td>(4) Maximum Force 100kips (c=10kips/s/in)</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>(4) Maximum Force 300kips (c=25kips/s/in)</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>(4) Maximum Force 300kips (c=25kips/s/in)</td>
</tr>
</tbody>
</table>

There is a reduction in the steel beam and concrete column sizes in the C-SMF with supplemental damping, compared to the conventional C-SMF. The beam sizes and hence the concrete column sizes in the conventional system are governed by system drift limitations. In the C-SMF with damping, the drift is no longer the governing criteria due to the damping provided by the viscous dampers. The beams are now governed by the gravity loads due to the long span conditions. Reduction in the steel beam sizes, results in smaller concrete column sizes to satisfy the strong column-weak beam requirements.
3.1.3. Code and Criteria

(1) The building shall be designed to remain essentially elastic at DE level seismic forces. The building shall be designed as Composite Special Moment Frames with viscous dampers.

(2) Nonlinear Response History Analysis is selected. Therefore, design coefficients and factors are as follows:

- Response Modification Coefficient: \( R = 1.0 \)
- Overstrength Factor: \( \Omega_0 = 1.0 \)
- Deflection Amplification Factor: \( C_d = 1.0 \)
- Redundancy Factor: \( \rho = 1.0 \)
- Strength Reduction Factor: \( \phi = 1.0 \)
- Member Criteria: \( DCR \leq 1.5 \)

(3) Seismic force shall not less than minimum force per 12.2.1 in ASCE7.

(4) Story Drift Limit (ASCE 7 Table 12.12-1)
\[ \Delta_a \leq 0.025h_{ix} \]

3.1.4. Ground Motion

The following seven records were used in the analysis. The available records were scaled and specially matched in accordance with ASCE 7-05. The scaling is consistent with the \( S_1 \) and \( S_2 \), in the response spectrum used in the Chapter 2 design earthquake. Therefore, the difference in member sizes and behavior of conventional C-SMF and C-SMF with viscous damper were based on the same grand motion parameters. The records were matched over a period range of 1 second to 6 second.

Table 3-2. Summary of selected horizontal ground motions

<table>
<thead>
<tr>
<th>Case</th>
<th>Event</th>
<th>Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ARL- NORTHRIDGE EQ 1/17/94, 12:31,</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>CDMG 24087 Arleta - Nordhoff Fire Sta, NGA 0949</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>CNP- NORTHRIDGE EQ 1/17/94, 12:31,</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>CANOGA PARK - TOPANGA CANYON (USC STATION 90053), NGA 0959</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>ECA3- Imperial Valley-06 1979-10-15 23:16,</td>
<td>1</td>
</tr>
<tr>
<td>6</td>
<td>USGS 5057 El Centro Array #3, NGA0178</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>GIL- LOMA PRIETA 10/18/89 00:05,</td>
<td>1</td>
</tr>
<tr>
<td>8</td>
<td>GILROY ARRAY #2, (CDMG STATION 47380), NGA 0766</td>
<td>2</td>
</tr>
<tr>
<td>9</td>
<td>NWH- NORTHRIDGE 01/17/94 12:31,</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>NEWHALL, (CDMG STATION 24279), NGA 1044</td>
<td>2</td>
</tr>
<tr>
<td>11</td>
<td>SYL- NORTHRIDGE, 1/17/94 12:31,</td>
<td>1</td>
</tr>
<tr>
<td>12</td>
<td>CDMG 24514 Sylmar - Olive View Med FF, NGA 1086</td>
<td>2</td>
</tr>
<tr>
<td>13</td>
<td>DEN TAPS-10- Denali, Alaska 2002-11-03,</td>
<td>1</td>
</tr>
<tr>
<td>14</td>
<td>Alyeska ps10 TAPS Pump Station #10, NGA 2114</td>
<td>2</td>
</tr>
</tbody>
</table>

1: Fault parallel to global X axis, Fault normal to global Y axis
2: Fault normal to global X axis, Fault parallel to global Y axis
3.2. Structural and Seismic Design Parameters

3.2.1. Structural Information

*Design Period*

Fundamental Period from Analysis  
\[ T_b = 1.13 \text{ sec (X)} \]  
\[ T_b = 1.00 \text{ sec (Y)} \]
3.3. Modeling and Analysis

3.3.1. 3D Model Conditions and Assignments

SAP2000 was used to generate 3D model for this design example. The foundation grade beams were not modeled. Instead of modeling the grade beam, the column bottom spring (Appendix A) was modeled to capture any non-linearity in the column.

**Support Conditions**

Column bottom spring was modeled instead of modeling the grade beam.

**Damper**

Viscous dampers are assigned by link element. The stiffness of driver brace is also assigned.
**Figure 3-7. Roof Framing Plan**

**Figure 3-8. 2\textsuperscript{nd} & 3\textsuperscript{rd} Floor Framing Plan**
Figure 3-9. 1st Floor Framing Plan

Figure 3-10. Framing Elevation (1 & 4)

Grade beams are not modelled in SAP model.

Column Bottom Spring

Isolated Footing

Column Bottom Spring

安心 Pin connection
3.4. Seismic Design Requirements for Building Structure

3.4.1. Base Shear Check
The minimum base shear requirement per ASCE 7 Chapter 18.2.2.1 is $0.75 \times Cs = 0.113 = 0.085$. Dynamic analysis resulted in $Cs_x = 0.34$ ($V_x = 1859\text{kip}$) and $Cs_y = 0.36$ ($V_y = 2014\text{kip}$) respectively. Therefore, the code limit is satisfied.

3.4.2. Irregularities Check
The building does not have any irregularities.

3.4.3. Redundancy Factor ($\rho$)
The $\rho$ is $1.0$ because this design example satisfies Condition (b) in ASCE 7 Section 12.3.4.2.

3.4.4. Drift Check
The building satisfied the design story drift requirements. The interstory drift ratio appeared to remain well within the code limit (see Table 3-3). The story drift values are half or less than those of composite RCS without viscous damper.

<table>
<thead>
<tr>
<th>X-dir</th>
<th>$\Sigma h_{sx}(\text{in})$</th>
<th>$h_{sx}(\text{in})$</th>
<th>$\Delta x(\text{in})$</th>
<th>$\Delta x/h_{sx}$</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Sigma h_{sx}(\text{mm})$</td>
<td>$h_{sx}(\text{mm})$</td>
<td>$\Delta x(\text{mm})$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof-3rd</td>
<td>540</td>
<td>180</td>
<td>1.9</td>
<td>0.010</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(13716)</td>
<td>(4572)</td>
<td>(47.6)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3rd-2nd</td>
<td>360</td>
<td>180</td>
<td>2.0</td>
<td>0.011</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(9144)</td>
<td>(4572)</td>
<td>(51.9)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2nd-1st</td>
<td>180</td>
<td>180</td>
<td>1.7</td>
<td>0.010</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(4572)</td>
<td>(4572)</td>
<td>(43.9)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Y-dir</th>
<th>$\Sigma h_{sy}(\text{in})$</th>
<th>$h_{sy}(\text{in})$</th>
<th>$\Delta y(\text{in})$</th>
<th>$\Delta y/h_{sy}$</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Sigma h_{sy}(\text{mm})$</td>
<td>$h_{sy}(\text{mm})$</td>
<td>$\Delta y(\text{mm})$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof-3rd</td>
<td>540</td>
<td>180</td>
<td>1.4</td>
<td>0.008</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(13716)</td>
<td>(4572)</td>
<td>(35.0)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3rd-2nd</td>
<td>360</td>
<td>180</td>
<td>1.7</td>
<td>0.010</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(9144)</td>
<td>(4572)</td>
<td>(43.9)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2nd-1st</td>
<td>180</td>
<td>180</td>
<td>1.7</td>
<td>0.009</td>
<td>$\leq$</td>
</tr>
<tr>
<td></td>
<td>(4572)</td>
<td>(4572)</td>
<td>(42.0)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.5. Member Design

3.5.1. Summary of Concrete Column Design

The concrete columns were designed in accordance with ACI 318-11. Biaxial loading effects were considered in the concrete column designs. As shown in the Table 3-4, the DCR for the columns is less than the limit of 1.5. The columns may have enough capacity because they are governed by strong column & weak beam requirement.

<table>
<thead>
<tr>
<th>Member</th>
<th>Direction</th>
<th>Level</th>
<th>Location</th>
<th>Section</th>
<th>Nu(kip)</th>
<th>Mu(kip-ft)</th>
<th>φMn(kip-ft)</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>X</td>
<td>3</td>
<td>2/B</td>
<td>Top 30”x30”(20)#10</td>
<td>106</td>
<td>448</td>
<td>1634</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>2/B</td>
<td>Bottom 30”x30”(20)#10</td>
<td>47</td>
<td>399</td>
<td>1287</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2/B</td>
<td>Top 30”x30”(20)#11</td>
<td>31</td>
<td>754</td>
<td>1742</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 30”x30”(20)#11</td>
<td>40</td>
<td>760</td>
<td>1716</td>
<td>0.44</td>
</tr>
<tr>
<td>C2</td>
<td>Y</td>
<td>3</td>
<td>2/A</td>
<td>Top 30”x30”(20)#10</td>
<td>-7</td>
<td>600</td>
<td>1541</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>2/A</td>
<td>Bottom 30”x30”(20)#10</td>
<td>9</td>
<td>493</td>
<td>1431</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2/A</td>
<td>Top 30”x30”(20)#11</td>
<td>17</td>
<td>1192</td>
<td>1820</td>
<td>0.66</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 30”x30”(20)#11</td>
<td>54</td>
<td>1056</td>
<td>1788</td>
<td>0.59</td>
</tr>
<tr>
<td>C3</td>
<td>Y</td>
<td>3</td>
<td>1/A</td>
<td>Top 30”x30”(16)#10</td>
<td>19</td>
<td>440</td>
<td>1262</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>1/A</td>
<td>Bottom 30”x30”(16)#10</td>
<td>81</td>
<td>413</td>
<td>1180</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>1/A</td>
<td>Top 30”x30”(16)#11</td>
<td>52</td>
<td>1016</td>
<td>1484</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom 30”x30”(16)#11</td>
<td>273</td>
<td>875</td>
<td>1564</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Table 3-4. Summary of Concrete Column Design
3.5.2. Summary of Steel Beam Design

The design of the steel beams was in accordance with ASCE 7, AISC 360 and AISC 341 and calculation procedure is the same as the design example in Chapter 2. As shown in the Table 3-5, every DCR for steel beam design is less than the limit of 1.5.

<table>
<thead>
<tr>
<th>Member</th>
<th>Floor</th>
<th>Location</th>
<th>Section</th>
<th>Mu</th>
<th>φMn</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bx</td>
<td>R</td>
<td>1/B-C</td>
<td>W21x62</td>
<td>436</td>
<td>479</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>2,3</td>
<td>2/B-C</td>
<td>W27x94</td>
<td>1215</td>
<td>998</td>
<td>1.22</td>
</tr>
<tr>
<td>Gy</td>
<td>R</td>
<td>A/1-2</td>
<td>W21x62</td>
<td>453</td>
<td>576</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td>2,3</td>
<td>A/1-2</td>
<td>W27x94</td>
<td>1495</td>
<td>1158</td>
<td>1.29</td>
</tr>
</tbody>
</table>

3.5.3. Summary of Damper Force

Table 3-6 summarizes the maximum force of viscous dampers. All of the damper forces are less than the capacity.

<table>
<thead>
<tr>
<th>Level</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>3rd</td>
<td>54</td>
<td>&lt; 100 OK</td>
</tr>
<tr>
<td>2nd</td>
<td>260</td>
<td>&lt; 300 OK</td>
</tr>
<tr>
<td>1st</td>
<td>260</td>
<td>&lt; 300 OK</td>
</tr>
</tbody>
</table>
3.5.4. Strong Column and Weak Beam (SCWB) Check

**Summary of Strong Column and Weak Beam Check**

The calculation procedure is same as the design example in Chapter 2. Figures 3-14 and 15 show only the summary. Every ratio of column to beam strength is greater than 1.0.

![Figure 3-14. Summary of SCWB in X-Direction](image)

![Figure 3-15. Summary of SCWB in Y-Direction](image)
3.5.5. Beam and Column Connection Design

The following calculations show the design of beam and column connections. The viscous damper is attached to the driver brace at one end with a fixed bolted connection, with the bolt pattern provided by the damper manufacturer. The opposite end of the driver brace is attached to a gusseted plate connection with a welded connection. On the opposite end of the damper, a pinned clevis-type connection is used to attach the damper to the bottom gusset plate, which is welded to the continuous steel beam and joint part. The configuration of a typical damper bay is shown in Figure 3-16. The design of driver brace and gusseted connection is beyond the scope of this design example.

Each connection shall be designed to achieve the connection strength governed by beam yielding plus additional stress due to the eccentric axial force of damper. In addition to the force shown in Chapter 2, an eccentric moment from viscous damper is applied to the beam-to-column joint resulting in additional joint shear. The beam-to-column joint calculation is repeated in its entirety in this chapter due to reduction in beam and column size.

![Figure 3-16. Configuration of a typical damper bay](image)

**Typical Connection Design (2nd Floor 2 x B X-direction)**

**Design Information**

**Geometric information**

- Span \( L = 50 \text{ ft} \)
- Floor height \( H = 15 \text{ ft} \)

**Beam section W27x94**

- Beam height \( d = 26.9 \text{ in} \)
- Beam width \( b_f = 10 \text{ in} \)
- Thickness of flange \( t_f = 0.745 \text{ in} \)
- Thickness of web \( t_w = 0.49 \text{ in} \)

**Column section 30”x30”**

- Column width \( b = 30 \text{ in} \)
- Column height \( h = 30 \text{ in} \)
- Specified concrete strength \( f'_c = 6000 \text{ psi} \)
Eccentric Moment from Viscous Damper

\[ P_{\text{max}} = 300 \text{kip} \] Maximum Axial Force of Viscous Damper

\[ \sin \theta_{\text{damper}} = \frac{15}{(25^2+15^2)^{0.5}} = 0.514 \]

\( e = 1.25 \text{ ft} \) Eccentric arm

\[ M_e = P_{\text{max}} \times \sin \theta_{\text{damper}} \times e = 192.94 \text{kips*ft} \]

Calculation of Joint Strength

Inner panel strength

\[ \phi V_{\text{in}} = \min(\phi_s \times (V_{\text{spn}} + V_{\text{icn}}), \phi_b \times (M_{\text{vb}} - V_b \times h) / dj) \]  

\( \phi_s = 0.85 \)

\( \phi_b = 0.75 \)

\( dj = d - tf = 26.2 \text{ in} \)

\[ V_{\text{spn}} = 0.6 \times F_{\text{ysp}} \times t_{\text{sp}} \times a_{\text{sp}} \times h \]  

\( F_{\text{ysp}} = 50 \text{ ksi} \)

\( t_{\text{sp}} = 0.49 \text{ in} \)

\( a_{\text{sp}} = 0.9 \)

\[ V_{\text{spn}} = 0.6 \times 50 \times 0.49 \times 0.9 \times h = 397 \text{ kip} \]

\[ V_{\text{icn}} = 1.7 \times \alpha_c \times (\sqrt{f_c'si}) \times b_i \times h / 4448.3 \times 25.4^2 \]  

\( \alpha_c = 1.0 \) for inner connection

\( \sqrt{f_c'si} = 6.48 \text{ ksi} \)

\( b_i = bf = 10.0 \text{ in} \)

\[ V_{\text{icn}} = 1.7 \times 1.0 \times 6.48 \times 10 \times h / 4448.3 \times 25.4^2 = 479 \text{ kip} \]

\[ V_{\text{icn}} = 479 \text{ kip} \]

\( dj = d - tf = 26.2 \text{ in} \)

\[ M_{\text{vb}} = C_{\text{cn}} \times h \times (1 - \beta_1 / 2) + h_{\text{vr}} \times (T_{\text{vm}} + C_{\text{vm}}) \]  

\( \alpha_{\text{cn}} = 2.5 \) for bearing regions with minimum re-bar ties

\( \beta_1 = 0.75 \) per concrete strength

\[ C_{\text{cn}} = \alpha_{\text{cn}} \times f_c' \times bf \times (\beta_1 \times h / 2) = 1688 \text{ kip} \]  

\( h_{\text{vr}} = h - 2 \times 3 \text{ in} = 24 \text{ in} \)

\( T_{\text{vm}} = 60 \text{ ksi} \times 0 \text{ in}^2 = 0 \text{ kip} \)

\( C_{\text{vm}} = 60 \text{ ksi} \times 0 \text{ in}^2 = 0 \text{ kip} \)

\[ M_{\text{vb}} = C_{\text{cn}} \times h \times (1 - \beta_1 / 2) + h_{\text{vr}} \times (T_{\text{vm}} + C_{\text{vm}}) = 2637 \text{ kips*ft} \]

\[ V_{\text{b}} = (M_{\text{b1}} + M_{\text{b2}}) / Lo \]

\( M_{\text{b1}} = 1402 \text{ kips*ft} \) Expected bare beam flexural strength

\( M_{\text{b2}} = 1402 \text{ kips*ft} \) Expected bare beam flexural strength

\( Lo = L - h = 47.5 \text{ ft} \)

\[ V_{\text{b}} = (M_{\text{b1}} + M_{\text{b2}}) / Lo = 59 \text{ kip} \]
**Inner panel strength**

\[ \phi V_{in} = \min(\phi_s \times (V_{spn} + V_{icn}), \phi_b \times (M_{vb} - V_b \times h / 12) / (dj / 12)) = 745 \text{ kip} \]

**Outer panel strength**

\[ \phi V_{on} = \phi_s \times 1.25 \times \alpha_c \times (\sqrt{f_{c'si}}) \times b_o \times h / 4448.3 \times 25.4^2 \]  
(Eq 18)

\[ \phi_s = 0.85 \]

\[ \alpha_c = 1.0 \text{ for inner connection} \]

\[ y = bf = 10.0 \text{ in} \]

\[ \alpha_s = 1.0 \text{ for Steel band plate with ties in the bearing region above and below beam} \]

[0.5 used in Chapter 2]

\[ X = h = 30 \text{ in for Band Plates} \]

\[ b_o = y + 2/3 \times \alpha_s \times X - bf = 20.0 \text{ in} \]

\[ \phi_s V_{on} = \phi_s \times 1.25 \times \alpha_c \times (\sqrt{f_{c'si}}) \times b_o \times h / 4448.3 \times 25.4^2 = 599 \text{ kip} \]

**Joint strength**

\[ k = 1.00 \]

\[ \phi V_n = k \times (\phi V_{in} + \phi V_{on}) = 1344 \text{ kip} \]  
(Eq 10)

**Calculation of joint force**

\[ \Sigma M_b = (M_{b1} + M_{b2}) + M_e = 2997 \text{ kips-ft} \]

\[ V_c = (M_{b1} + V_b \times (h / 2) + M_{b2} + V_b \times (h / 2)) / H = 197 \text{ kip} \]

\[ V_j = \Sigma M_b / dj - V_c = 1176 \text{ kip} \]  
(Eq 11)

**Capacity check**

\[ \text{DCR} = V_j / \phi V_n = 0.88 \]

\[ V_j < \phi V_n \text{ OK} \]

**Summary of Connection Design**

Table 3-7 shows the summary of connection design. As shown in the table, every DCR for connection design is less than the limit of 1.0. Due to the additional joint shear from damper moment, ties are provided in the band plate region.

<table>
<thead>
<tr>
<th>Level</th>
<th>Location</th>
<th>Direction</th>
<th>$V_{spn}(\text{kip})$</th>
<th>$V_{icn}(\text{kip})$</th>
<th>$\alpha$</th>
<th>$V_{on}(\text{kip})$</th>
<th>$\phi V_{in}(\text{kip})$</th>
<th>$\phi V_{on}(\text{kip})$</th>
<th>$\phi V_n(\text{kip})$</th>
<th>$V_j(\text{kip})$</th>
<th>DCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>RF</td>
<td>GL2 x GLB</td>
<td>X</td>
<td>324</td>
<td>395</td>
<td>0.50</td>
<td>352</td>
<td>611</td>
<td>300</td>
<td>911</td>
<td>842</td>
<td>0.92</td>
</tr>
<tr>
<td>2nd</td>
<td>GL2 x GLB</td>
<td>X</td>
<td>397</td>
<td>479</td>
<td>1.00</td>
<td>705</td>
<td>745</td>
<td>599</td>
<td>1344</td>
<td>1178</td>
<td>0.88</td>
</tr>
<tr>
<td>RF</td>
<td>GL2 x GLA</td>
<td>Y</td>
<td>324</td>
<td>395</td>
<td>0.50</td>
<td>352</td>
<td>611</td>
<td>300</td>
<td>911</td>
<td>835</td>
<td>0.92</td>
</tr>
<tr>
<td>2nd</td>
<td>GL2 x GLA</td>
<td>Y</td>
<td>397</td>
<td>479</td>
<td>1.00</td>
<td>705</td>
<td>745</td>
<td>599</td>
<td>1344</td>
<td>1160</td>
<td>0.86</td>
</tr>
</tbody>
</table>
### 3.5.6. Connection Detail Design

The following calculations show the detailed design of beam & column interior and exterior connections. Each connection detail shall be designed to achieve the inner panel and outer panel strength of the connection. The connection was designed using the ASCE Pre-Standard.

**CONNECTION DETAIL DESIGN (Interior - 2nd Floor)**

![Figure 3-17. Elevation View](image1)

![Figure 3-18. Plan view](image2)

**Design Information**

**Design Stress (Requirement)**

- **Outer Panel Strength**
  - \( V_{on} = 705 \text{ kip from Table 3-7} \)
- **Inner Panel Strength**
  - \( V_{icn} = 479 \text{ kip from Table 3-7} \)

**Configuration**

- **Concrete Column Size**
  - \( b = 30 \text{ in} \)
- **Concrete Column Size**
  - \( h = 30 \text{ in} \)
- **Band Plate Height**
  - \( d_{bp} = 8 \text{ in} \)

**Tie Bar**

- (4) #5 @4” OC EW
- **Area of Tie Bar**
  - \( A_{tie} = 0.31 \text{ in}^2 \)
- **Tie Bar Legs legs**
  - 4
- **Tie Bar Space**
  - \( s_h = 4 \text{ in} \)
**Band Plate**

Yield Strength of Steel Band Plate \( F_{ybp} = 50 \text{ ksi} \)

Tensile Strength of Steel Band Plate \( F_{ubp} = 65 \text{ ksi} \)

Band Plate Thickness \( t_{bp} = 1.00 \text{ in} \) [0.875 in Chapter 2]

**Face Bearing Plate**

Yield Strength of Face Bearing Plate \( F_{yp} = 50 \text{ ksi} \)

Tensile Strength of Face Bearing Plate \( F_{up} = 65 \text{ ksi} \)

Face Bearing Plate Thickness \( t_{p} = 0.875 \text{ in} \)

**Longitudinal Column Re-Bars**

(20)#11

Number of Vertical Bar \( n_v = 20 \)

Vertical Bar Diameter \( d_b = 1.41 \text{ in} \)

Yield Strength of Longitudinal Column Bars \( F_{yr} = 60 \text{ ksi} \) \( (F_{yr,si} = 420 \text{ MPa}) \)

**Steel Beam**

W27x94

Depth of Beam \( d = 26.9 \text{ in} \)

Width of Beam \( b_f = 10 \text{ in} \)

Thickness of Web \( t_{sp} = 0.49 \text{ in} \)

Thickness of Flange \( t_f = 0.745 \text{ in} \)

Effective Joint Depth \( d_j = d - t_r = 26.15 \text{ in} \)

Yield Strength of Web \( F_{ysp} = 50 \text{ ksi} \)

Yield Strength of Flange \( F_{yf} = 50 \text{ ksi} \)

The Flange Width of Steel Band Plate \( b'_p = 10 \text{ in} \)

**Check of Connection Detail**

**Tie Bar Check**

1) Maximum Spacing Between Horizontal Ties

\[ \min(0.25 \times d_j, 0.25 \times h) = 6.54 \text{ in} \]

OK, 4” provided
2) Volumetric Ratio of Horizontal Rebar
\[
\rho_{s,\text{require}} = 0.01, \quad V_{s,\text{require}} = \rho_{s,\text{require}} \times (dj \times h \times b) = 267.8 \text{ in}^3
\]
\[
V_{s,\text{design}} = \text{legs} \times A_{\text{tie}} \times \{(b - 2 \times 2\text{"}) + (h - 2 \times 2\text{")}\} \times dj / s_h = 421.5 \text{ in}^3
\]
\[\text{OK, } V_{s,\text{design}} > V_{s,\text{require}}\]

**Concrete Confinement Check**

1) Band Plate Extension
\[
0.25 \times dj = 6.54 \text{ in}
\]
\[\text{OK, } d_{bp} = 8 \text{ in} > 6.54 \text{ in}\]

2) Minimum Volume
\[
V_{bp,\text{min}} = 0.03 \times dj \times h \times b = 706 \text{ in}^3
\]
\[
V_{bp} = t_{bp} \times d_{bp} \times 2 \times (b + h) = 960 \text{ in}^3
\]
\[\text{OK, } V_{bp} > V_{bp,\text{min}}\]

3) Band Plate Area
\[\text{ASCE Eq 19}\]
\[
A_{bp,\text{min}} = V_{\text{on}} / F_{ybp} = 14.1 \text{ in}^2
\]
\[\text{OK, } A_{bp} > A_{bp,\text{min}}\]

**Longitudinal Column Re-Bars Check**

\[
d_{b,\text{max}} = \frac{dj' \times 420}{20F_{yr}} \text{ (ASCE Eq 20)}
\]
\[dj' = 1.25 \times dj = 32.7 \text{ in} \text{ (Steel Band Plate Are Used)}\]
\[
d_{b,\text{max}} = dj' \times 420 \text{ MPa} / (20 \times F_{yr,si}) = 1.64 \text{ in}
\]
\[\text{OK, } d_b = 1.41 \text{ in} < d_{b,\text{max}}\]

**Steel Beam Flanges Check**

\[t_{f,\text{min}} = 0.30 \sqrt{\frac{b_f t_{sp} d' F_{ysp}}{h F_{yf}}} \text{ (ASCE Eq 21)}
\]
\[t_{f,\text{min}} = 0.30 \times \sqrt{(b \times t_{sp} \times d' \times F_{ysp})/(h \times F_{yf})} = 0.63 \text{ in}
\]
\[\text{OK, } t_{f,\text{provided}} = 0.754 \text{ in} > t_{f,\text{min}}\]

**Minimum Thickness of Band Plate Check**

\[t_{pp,\text{min}} = 0.12 \sqrt{\frac{V_{on} b_p}{d_{bp} F_{ybp}}} \text{ (ASCE Eq 25)}
\]
\[\text{Depth of Steel Band Plate } d_{bp} = 8.0 \text{ in}
\]
\[t_{bp,\text{min}} = 0.12 \times \sqrt{(V_{on} \times b'_p)/(d_{bp} \times F_{ybp})} = 0.56 \text{ in}\]
\[ t_{bp,min} = \frac{\sqrt{3}}{2b'p F_{ubp}} \cdot V_{on} \]  

(ASCE Eq 26)

\[ t_{bp,min} = \left(\frac{\sqrt{3}}{2 \times b'p \times F_{ubp}}\right) \times V_{on} = 0.94 \text{ in} \]

**OK, \( t_{bp,provided} = 1.00 \text{ in} > t_{bp,min} \)**

**Face Bearing Plates Check**

\[ t_{p,min} = \frac{\sqrt{3}}{2b_f F_{up}} \cdot V_{icn} \]  

(ASCE Eq 22)

\[ t_{p,min} = \left(\frac{\sqrt{3}}{2 \times b_f \times F_{up}}\right) \times V_{icn} = 0.64 \text{ in} \]

\[ t_{p,min} = 0.20 \cdot \sqrt{\frac{V_{icn} b_f}{F_{yp} d_w}} \]  

(ASCE Eq 23)

\[ d_w = (d - 2 \times t_f) = 25.41 \text{ in} \]

\[ t_{p,min} = 0.20 \times \sqrt{\left(\frac{V_{icn} \times b_f}{F_{yp} \times d_w}\right)} = 0.39 \text{ in} \]

\[ t_{p,min} = \frac{b_f}{22} \]  

(ASCE Eq 24)

\[ t_{p,min} = b_f / 22 = 0.45 \text{ in} \]

**OK, \( t_{p,provided} = 0.875 \text{ in} > t_{p,min} \)**


**Design Information**

**Design Stress**
- Outer Panel Strength \( V_{on} = 352 \text{ kip from Table 3-7} \)
- Inner Panel Strength \( V_{icn} = 395 \text{ kip from Table 3-7} \)

**Configuration**
- Concrete Column Size \( b = 30 \text{ in} \)
- Concrete Column Size \( h = 30 \text{ in} \)
- Band Plate Height \( d_{bp} = 6.5 \text{ in} \)

**Tie Bar**
- (4) \#5 @4” OC EW
- Area of Tie Bar \( A_{tie} = 0.31 \text{ in}^2 \)
- Tie Bar Legs \( \text{legs} = 4 \)
- Tie Bar Space \( s_{h} = 4 \text{ in} \)
Band Plate
Yield Strength of Steel Band Plate \( F_{ybp} = 50 \text{ ksi} \)
Tensile Strength of Steel Band Plate \( F_{ubp} = 65 \text{ ksi} \)
Band Plate Thickness \( t_{bp} = 0.875 \text{ in} \)

Face Bearing Plate
Yield Strength of Face Bearing Plate \( F_{yp} = 50 \text{ ksi} \)
Tensile Strength of Face Bearing Plate \( F_{up} = 65 \text{ ksi} \)
Face Bearing Plate Thickness \( t_{p} = 0.875 \text{ in} \)

Longitudinal Column Re-Bars
(20)#10
Number of Vertical Bar \( n_v = 20 \)
Vertical Bar Diameter \( d_b = 1.27 \text{ in} \)
Yield Strength of Longitudinal Column Bars \( F_{yr} = 60 \text{ ksi} \) \((F_{yr_{-si}} = 420 \text{ MPa})\)

Steel Beam
W21x62
Depth of Beam \( d = 21 \text{ in} \)
Width of Beam \( b_f = 8.24 \text{ in} \)
Thickness of Web \( t_{sp} = 0.4 \text{ in} \)
Thickness of Flange \( t_f = 0.615 \text{ in} \)
Effective Joint Depth \( d_j = d - t_r = 20.39 \text{ in} \)
Yield Strength of Web \( F_{ysp} = 50 \text{ ksi} \)
Yield Strength of Flange \( F_{yf} = 50 \text{ ksi} \)
The Flange Width of Steel Band Plate \( b'_{p} = 8.24 \text{ in} \)

Check of Connection Detail

Tie Bar Check
1) Maximum Spacing Between Horizontal Ties
\( \min(0.25 \times d_j, 0.25 \times h) = 5.10 \text{ in} \)
   \text{OK, 4” provided}
DESIGN OF COMPOSITE RCS SPECIAL MOMENT FRAME

2) Volumetric Ratio of Horizontal Rebar
\[ \rho_{s,\text{require}} = 0.01, \quad V_{s,\text{require}} = \rho_{s,\text{require}} \times (dj \times h \times b) = 183.5 \text{ in}^3 \]
\[ V_{s,\text{design}} = \text{legs} \times \text{As} \times \{ (b - 2 \times 2") + (h - 2 \times 2") \} \times \frac{dj}{sh} = 328.7 \text{ in}^3 \]
\[ \text{OK, } V_{s,\text{design}} > V_{s,\text{require}} \]

**Concrete Confinement Check**

1) Band Plate Extension
\[ 0.25 \times dj = 5.10 \text{ in} \]
\[ \text{OK, } d_{bp} = 6.5 \text{ in} > 5.10 \text{ in} \]

2) Minimum Volume
\[ V_{bp,\text{min}} = 0.03 \times dj \times h \times b = 550 \text{ in}^3 \]
\[ V_{bp} = t_{bp} \times d_{bp} \times 2( b + h ) = 683 \text{ in}^3 \]
\[ \text{OK, } V_{bp} > V_{bp,\text{min}} \]

3) Band Plate Area
\[ A_{bp,\text{min}} = \frac{V_{on}}{F_{ybp}} \] (ASCE Eq 19)
\[ A_{bp} = t_{bp} \times d_{bp} \times 2 = 11.4 \text{ in}^2 \]
\[ \text{OK, } A_{bp} > A_{bp,\text{min}} \]

**Longitudinal Column Re-Bars Check**

\[ d_{b,\text{max}} = \frac{dj(420)}{20F_{yr}} \] (ASCE Eq 20)
\[ dj' = 1.25 \times dj = 25.5 \text{ in} \text{ (Steel Band Plate Are Used)} \]
\[ d_{b,\text{max}} = dj' \times 420 \text{ MPa} / (20 \times F_{yr,si}) = 1.28 \text{ in} \]
\[ \text{OK, } d_{b} = 1.27 \text{ in} < d_{b,\text{max}} \]

**Steel Beam Flanges Check**

\[ t_{f,\text{min}} = 0.30 \sqrt{\frac{b'd_{sp}d_{Ysp}}{F_{ysp}h_{Fyf}}} \] (ASCE Eq 21)
\[ t_{f,\text{min}} = 0.30 \sqrt{(b \times t_{sp} \times d \times F_{ysp})/(h \times F_{yf})} = 0.46 \text{ in} \]
\[ \text{OK, } t_{f,\text{provided}} = 0.615 \text{ in} > t_{f,\text{min}} \]

**Minimum Thickness of Band Plate Check**

\[ t_{bp,\text{min}} = 0.12 \sqrt{\frac{V_{on}b_p}{d_{bp}F_{ybp}}} \] (ASCE Eq 25)
\[ \text{Depth of Steel Band Plate } d_{bp} = 0.25 \times dj = 5.10 \text{ in} \]
\[ t_{bp,\text{min}} = 0.12 \sqrt{(V_{on} \times b'_p)/(d_{bp} \times F_{ybp})} = 0.40 \text{ in} \]
\[ t_{bp,\text{min}} = \frac{\sqrt{3}}{2 \cdot b_p \cdot F_{ubp}} \cdot V_{on} \]  

(ASCE Eq 26)

\[ t_{bp,\text{min}} = \left( \sqrt{3} \right) / \left( 2 \times b'_p \times F_{ubp} \right) \times V_{on} = 0.57 \text{ in} \]

\text{OK, } t_{bp,\text{provided}} = 0.875 \text{ in} > t_{bp,\text{min}}

\text{Face Bearing Plates Check}

\[ t_{p,\text{min}} = \frac{\sqrt{3}}{2 \cdot b_f \cdot F_{up}} \cdot V_{ic} \]  

(ASCE Eq 22)

\[ t_{p,\text{min}} = \left( \sqrt{3} \right) / \left( 2 \times b_f \times F_{up} \right) \times V_{ic} = 0.64 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \times \sqrt{\frac{V_{icn} \cdot b_f}{F_{yp} \cdot d_w}} \]  

(ASCE Eq 23)

\[ d_w = (d - 2 \times t_f) = 19.77 \text{ in} \]

\[ t_{p,\text{min}} = 0.20 \times \sqrt{\frac{V_{icn} \times b_f}{F_{yp} \times d_w}} = 0.36 \text{ in} \]

\[ t_{p,\text{min}} = \frac{b_f}{22} \]  

(ASCE Eq 24)

\[ t_{p,\text{min}} = b_f / 22 = 0.37 \text{ in} \]

\text{OK, } t_{p,\text{provided}} = 0.875 \text{ in} > t_{p,\text{min}}
3.6. Typical Details

Typical details for composite RCS beam-column connections using dampers are included in this section. Cast in place concrete construction was considered while developing these details. Other construction procedures including pre-cast concrete may be considered.

2nd & 3rd Floor

Figure 3-21. Beam & Column Connection Detail at the 2nd & 3rd Floors (Cast in Place)
Figure 3-22. Beam & Column Connection Detail at Roof Floors (Cast in Place)
3.7. Comparison of Structural Performance

The comparison between Composite RCS and Composite RCS with Damper is shown in this section. Configuration of each building is the same as design example.

**Comparison of Structural System**

Table 3-8 summarizes and compares the typical member sizes of for the RCS frame with and without dampers. As indicated, several of the beam and column sizes are smaller for the frame with dampers.

<table>
<thead>
<tr>
<th>Seismic Force Resistance System</th>
<th>Level</th>
<th>Column (Concrete)</th>
<th>Beam (Steel)</th>
<th>Damping Device (Viscous Damper)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>X(Direction)</td>
<td>Y(Direction)</td>
</tr>
<tr>
<td>Composite RCS</td>
<td>3rd</td>
<td>32&quot; x 32&quot;</td>
<td>W21 x 62</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>32&quot; x 32&quot;</td>
<td>W30 x 108</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>32&quot; x 32&quot;</td>
<td>W30 x 108</td>
<td></td>
</tr>
<tr>
<td>Composite RCS w/ Damper</td>
<td>3rd</td>
<td>30&quot; x 30&quot;</td>
<td>W21 x 62</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>30&quot; x 30&quot;</td>
<td>W27 x 94</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>30&quot; x 30&quot;</td>
<td>W27 x 94</td>
<td></td>
</tr>
</tbody>
</table>

**Comparison of Structural Performance**

Table 3-9 summarizes the story drift ratios and floor accelerations that were calculated for each of the designs under the design level earthquake ground motions.

<table>
<thead>
<tr>
<th>Seismic Force Resistance System</th>
<th>Level</th>
<th>Inter Story Drift (%)</th>
<th>Floor Acceleration (g)</th>
<th>R factor</th>
<th>Immediate Occupancy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>X(Direction)</td>
<td>Y(Direction)</td>
<td>X(Direction)</td>
<td>Y(Direction)</td>
</tr>
<tr>
<td>Composite RCS</td>
<td>3rd</td>
<td>2.10</td>
<td>1.50</td>
<td>1.57</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>2.30</td>
<td>1.80</td>
<td>0.80</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>1.90</td>
<td>1.60</td>
<td>0.63</td>
<td>0.67</td>
</tr>
<tr>
<td>Composite RCS w/ Damper</td>
<td>3rd</td>
<td>1.00</td>
<td>0.80</td>
<td>0.65</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>1.10</td>
<td>1.00</td>
<td>0.45</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>1.00</td>
<td>0.90</td>
<td>0.43</td>
<td>0.44</td>
</tr>
</tbody>
</table>

**Conclusion**

This example illustrates that the introduction of dampers to conventional composite RCS frames increases the seismic performance of the lateral system and further reduces the structural steel costs.
REFERENCES

ACI (2011), *Building Code Requirements for Structural Concrete*, ACI 318-11, American Concrete Institute.


ASCE (2015), *Pre-Standard for the Design of Moment Connections between Steel Beams and Concrete Columns*, ASCE 2015 draft, American Society of Civil Engineers.


In this appendix, nonlinear analysis assumptions for a SAP2000 model are described, and analysis results are compared with data from a pseudo-dynamic test of a full-scale composite frame, which was carried out at the National Center for Research in Earthquake Engineering (NCREE) by Cordova and Deierlein of Stanford University and Tsai and colleagues of National Taiwan University (Cordova and Deierlein 2005).

**TEST FRAME**

Schematic of the test model is presented in Figure A1. The United States Customary Units (US Units) are used to model the Test Frame in SAP, Table A1 summarizes the dimensions in US Units, equivalent US rolled steel shapes, and equivalent material properties used in the model.

![Figure A1: Test Frame Schematic Model](image)

**Table A1: Frame Member Properties (Based on Measured Strength)**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Steel Beams</th>
<th>RC Column</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Section</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1st</td>
<td>W24X68 (H600x200)</td>
<td>26”x26” (650 mm X 650 mm)</td>
</tr>
<tr>
<td></td>
<td>Fy= 60 ksi (≈420MPa)</td>
<td>fc’=10,960 psi (≈76 MPa)</td>
</tr>
<tr>
<td>2nd</td>
<td>W21X55 (H500x200)</td>
<td>26”x26” (650 mm X 650 mm)</td>
</tr>
<tr>
<td></td>
<td>Fy= 72 ksi (≈500MPa)</td>
<td>fc’=8,364 psi (≈58 MPa)</td>
</tr>
<tr>
<td>3rd</td>
<td>W16X36 (H396x199)</td>
<td>26”x26” (650 mm X 650 mm)</td>
</tr>
<tr>
<td></td>
<td>Fy= 60 ksi (≈420MPa)</td>
<td>fc’=8,364 psi (≈58 MPa)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Non-linear Model using SAP2000

Figure A2 through A5 represent the typical modeling assumption in SAP 200.

1. Elastic stick model with nonlinear spring at the end of member is applied in SAP2000. Material properties in SAP2000 are shown in Table A1.
2. Multiple-springs in one connection are not allowed in SAP2000. Therefore, Shear Panel Spring and Vertical Bearing Spring in beam-to-column connection shall be combined into one link spring. The concept of modification is shown in Figure A4.
3. The hinge properties in SAP2000 are shown in Figure A5.
4. The beam stiffness shall be the average stiffness of bare steel beam and composite beam.

Typical modeling assumption

![Diagram of Beam-Column Connections](image)

Where:

\[ K_{rot} = \frac{3E_{l_{eff}}}{L_i} \]

\[ \frac{E_{l_{eff}}}{E_{l_{g, tr}}} = 0.4 + 0.6 \frac{P}{2.4P_b} \leq 0.9 \]

**Figure A2: Modeling of Beam-Column Connections**

![Diagram of Column Bottom Spring](image)

**Figure A3: Modeling of Column Bottom Spring**
**Beam Stiffness (in SAP2000)**

\[
E \left( \frac{\alpha + 1.0}{2} \right) l_y
\]

\( \alpha \); Slab Effect (Positive Bending)
DYNAMIC ANALYSIS

Seismic Force

1. 50% CHANCE OF EXCEEDING IN 50 YEARS (50/50)
   1999 CHI-CHI (TCU082), Sa (T₁)=0.408g

2. 10% CHANCE OF EXCEEDING IN 50 YEARS (10/50)
   1989 LOMA PRIETA (LP89 G04), Sa (T₁)=0.68g

3. 2% CHANCE OF EXCEEDING IN 50 YEARS (2/50)
   1999 CHI-CHI (TCU082), Sa (T₁)=0.92g

Calculation Method for Dynamic Analysis

DIRECT INTEGRATION METHOD
NON LINEAR ANALYSIS
DAMPING h=2%
MOMENT-ROTATION RELATION AT BEAM-COLUMN JOINT

Figure A6: Response Spectrum

Figure A7: Moment Rotation Relationship
COMPARISON OF DYNAMIC ANALYSIS BETWEEN FULL SCALE COMPOSITE TEST AND SAP NONLINEAR MODEL

Dynamic analysis results of SAP were overlapped with results of Full scale composite test frame. Roof Disp, Base shear peak IDR and peak story shear are shown from Figure A8 to A11.

Figure A8: Dynamic Analysis Result in 50/50 event
Figure A9: Dynamic Analysis Result in 10/50-1a event
Figure A10: Dynamic Analysis Result in 50/50-1b event
Figure A11: Dynamic Analysis Result in 2/50 event
CONCLUSION

The results in a 50/50 ground motion (chance of 50% exceedance in 50 years) and 10/50 ground motion show reasonable conformation with Full-scale composite test frame results. Likewise, the displacement results in the 2/50 event show reasonable agreement up with the Full-Scale frame test until the peak drifts reach about 3% to 4% drift. Beyond 4% drift, the beam flanges experience degradation due to local buckling that is not captured well by the analysis model. Nevertheless, provided that inelastic drifts are limited to about 4%, which is required by most building codes, then the SAP2000 analysis is reasonably accurate for design.
APPENDIX B
LINEAR MODELING USING SAP2000

The purpose of this section is to describe the modeling assumptions to create a Linear Analysis model in SAP2000.

For linear analysis, an elastic stick model is adopted and response modification factors are applied to the beam and columns. The beam-to-column connections are modified as rigid connections with centerline dimensions, neglecting finite joint size and link elements. Column bottom springs remain the same as the nonlinear model. See the Parametric Study Appendix C for Response Modification Factors to be used in the linear analysis.

GENERAL OUTLINE OF MODIFICATION TO SAP NONLINEAR MODEL TO GENERATE LINEAR ANALYSIS MODEL

Figure B1 to B3 exhibit the modifications to Nonlinear Model. For a detailed description of the Nonlinear Model, refer to the Appendix A - Nonlinear Analysis of Full Scale composite Test Frame Assembly using SAP2000.

Figure B1: Modifications to the Beam-Column Connection
Design of Composite RCS Special Moment Frames

\[ K_{rot} = \frac{3EI_{eff}}{Li} \]

\[ EI_{eff} = 0.4 + 0.6 \frac{P}{2AP_b} \leq 0.9 \]

WHERE:
- \( EI_{eff} \) = effective stiffness of the cracked reinforced concrete section
- \( EI_{gy} \) = transformed stiffness of the gross reinforced concrete section
- \( P \) = expected axial load in reinforced concrete column
- \( P_b \) = balanced axial load taken from the RC column P-M interaction diagram
- \( \phi_b EI_b \) = equivalent linear stiffness of the steel beam
- \( \phi_c EI_c \) = equivalent linear stiffness of the concrete column

Figure B2: Modifications to Column Bottom Spring Beam & Column Joint

Figure B3: Modification Non-linear Spring at Beam-to-Column Connection
APPENDIX C
PARAMETRIC STUDY

There has been a significant improvement in computing power in the last decade, which allows for efficient use of nonlinear analysis procedures. However, nonlinear procedures are still perceived to be cumbersome and time consuming in the design industry.

The purpose of this parametric study is to determine the equivalent stiffness modification parameters to be used in performing linear analysis procedures in SAP2000. Nonlinear Analysis using SAP2000 is discussed in Appendix A.

PARAMETRIC MODELS

Eight different models were created and studied to determine the response modification parameters. The “Nonlinear Analysis of Full Scale Composite Test Frame Assembly using SAP2000” in Appendix A was considered as a baseline or “Control” case. The following geometric and stiffness modifications were made to the Control case.

1. Length of column = 2 x Control. **05
2. Length of beam = 2 x Control. **20
3. Stiffness of column = 2 x Control. 05**
4. Stiffness of beam = 2 x Control. 20**

Table C1 summarizes the modifications. Model 1010 is same model as Full-Scale RCS Test Frame (Control case). The column length in Model **05 series is twice the length of Control. The beam length in Model **20 series is twice the length of Control. The column stiffness in Model 05** series is twice the stiffness of Control. The beam stiffness in Model 20** series is twice the stiffness of Control.

Nonlinear pushover analysis was performed on each model and the results were plotted against the control. Each model was then run for 2 linear cases, case 1 and 2. See the Analysis and Results Section for further discussion.

Table C2 summarizes the modifications made to the beam and column sections to achieve the increased stiffness.
Table C1: Summary of Models used for Parametric Study

<table>
<thead>
<tr>
<th>Model 0505</th>
<th>Model 1005</th>
<th>Model 2005</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Diagram of Model 0505]</td>
<td>![Diagram of Model 1005]</td>
<td>![Diagram of Model 2005]</td>
</tr>
<tr>
<td>$L_g = 1.0 \times L_{gt}$</td>
<td>$I_g = 1.0 \times I_{gt}$</td>
<td>$L_c = 2.0 \times L_{ct}$</td>
</tr>
<tr>
<td>$L_c = 2.0 \times L_{ct}$</td>
<td>$I_c = 2.0 \times I_{ct}$</td>
<td>$L_g = 1.0 \times L_{gt}$</td>
</tr>
</tbody>
</table>

Table C2: Summary of Beams and Column Sizes used for Parametric Study

<table>
<thead>
<tr>
<th>Member</th>
<th>Control</th>
<th>Parametric Model (2.0 $I_g$, 2.0 $I_c$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>H396x199(W16x26), $l=488$, Slab Effect $\alpha_b=1.5$</td>
<td>W18x55, $l=890$, Slab Effect $\alpha_b=1.4$</td>
</tr>
<tr>
<td></td>
<td>H500x200(W21x55), $l=1140$, Slab Effect $\alpha_b=1.4$</td>
<td>W21x101, $l=2420$, Slab Effect $\alpha_b=1.3$</td>
</tr>
<tr>
<td></td>
<td>H600x200(W24x68), $l=1830$, Slab Effect $\alpha_b=1.3$</td>
<td>W27x102, $l=3620$, Slab Effect $\alpha_b=1.2$</td>
</tr>
<tr>
<td>Column</td>
<td>650x650(26&quot;x26&quot;), $l=1.5 \times 10^6$</td>
<td>775x775(31&quot;x31&quot;), $l=3.0 \times 10^5$</td>
</tr>
</tbody>
</table>

$\alpha_b =$ Average Modifier of Positive(as Composite Beam) and Negative Bending
ANALYSIS AND RESULTS

Pushover Analysis

Pushover analysis was performed on all eight models to study their behavior and compare them to the Control. Comparison of pushover curves for each model is shown in Figure C1 below. Model **05 series has approximately half the initial stiffness and half the strength compared to the Control. Model 0510 & 2010 have a high initial stiffness and about 20% and 30% higher strength compared to the Control. Model **20 series has low initial stiffness and approximately same strength compared to the Control. From comparing these results, it is evident that the eight model cases cover all anticipated variations/modifications to the test frame assembly.

![Figure C1: Comparison of Pushover Analysis](image)

Results

Table C3 summarizes the characteristics and findings of each Model as follows

1. The Model 20** series have story collapse behavior because of the weaker strength of column compared to the beam.
2. The maximum base shear of each Model **05 is almost half that of **10 series & **20 series corresponding to the column length.
3. Model 05** series and 20** series have about 20% and 30% higher strength than that of 10** series respectively.
Shear strength of the first story depends on flexural strength of the column and beam and flexural (rotational) stiffness of column bottom.

### Table C3: Summary of Characteristics of 8 Nonlinear Models

<table>
<thead>
<tr>
<th></th>
<th>$l_g=1.0x l_{gt}$</th>
<th>$l_g=1.0x l_{gt}$</th>
<th>$l_g=2.0x l_{gt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l_c=2.0x l_{ct}$</td>
<td>$l_c=1.0x l_{ct}$</td>
<td>$l_c=1.0x l_{ct}$</td>
</tr>
<tr>
<td>Model 0505</td>
<td>1.71~2.31</td>
<td>0.93~1.21</td>
<td>0.47~0.69</td>
</tr>
<tr>
<td>Model 1005</td>
<td>0.93~1.21</td>
<td>0.47~0.69</td>
<td>0.67</td>
</tr>
<tr>
<td>Model 2005</td>
<td>0.47~0.69</td>
<td>0.67</td>
<td>0.50</td>
</tr>
<tr>
<td>Base Shear Strength (kN)</td>
<td>2186</td>
<td>1648</td>
<td>1948</td>
</tr>
<tr>
<td>Ratio to Model 1010</td>
<td>0.67</td>
<td>0.50</td>
<td>0.59</td>
</tr>
<tr>
<td>Model 0510</td>
<td>1.84~2.47</td>
<td>1.00~1.29</td>
<td>0.50~0.76</td>
</tr>
<tr>
<td>Model 1010</td>
<td>1.00~1.29</td>
<td>0.50~0.76</td>
<td>1.31</td>
</tr>
<tr>
<td>Model 2010</td>
<td>0.50~0.76</td>
<td>1.31</td>
<td>1.00</td>
</tr>
<tr>
<td>Base Shear Strength (kN)</td>
<td>4288</td>
<td>3278</td>
<td>3846</td>
</tr>
<tr>
<td>Ratio to Model 1010</td>
<td>1.31</td>
<td>1.00</td>
<td>1.17</td>
</tr>
<tr>
<td>Model 0520</td>
<td>1.96~2.61</td>
<td>1.06~1.36</td>
<td>0.53~0.79</td>
</tr>
<tr>
<td>Model 1020</td>
<td>1.06~1.36</td>
<td>0.53~0.79</td>
<td>1.29</td>
</tr>
<tr>
<td>Model 2020</td>
<td>0.53~0.79</td>
<td>1.29</td>
<td>1.00</td>
</tr>
<tr>
<td>Base Shear Strength (kN)</td>
<td>4216</td>
<td>3280</td>
<td>3976</td>
</tr>
<tr>
<td>Ratio to Model 1010</td>
<td>1.29</td>
<td>1.00</td>
<td>1.21</td>
</tr>
<tr>
<td>Weak Beam &amp; Strong Column</td>
<td>↑</td>
<td>↑</td>
<td>↑</td>
</tr>
<tr>
<td>Weak Beam &amp; Strong Column</td>
<td>↑</td>
<td>↑</td>
<td>↑</td>
</tr>
<tr>
<td>Strong Beam &amp; Weak Column</td>
<td>↑</td>
<td>↑</td>
<td>↑</td>
</tr>
</tbody>
</table>
Linear Analysis

Analysis

Two linear cases of each of the eight Nonlinear Models were created and studied. In each case, stiffness modification factors were used to obtain behavior that is as close as possible to the nonlinear analysis results.

Case 1

Stiffness modification factor for the beam ($\alpha_b$) varies between 1.3 to 1.5, and stiffness modification factor for the column ($\alpha_c$) is 0.5. See Table C4 for further clarification. Variable $\alpha_b$ is based on a nonlinear analysis assignment (primarily due to composite action) and variable $\alpha_c$ for the column is based on the performance of column yielding and cracking due to stiff beam.

Case 2

Stiffness modification factor for the beam ($\alpha_b$) is 1.0, and the stiffness modification factor for the column ($\alpha_c$) is 0.7. See Table C4 for further clarification. Here, $\alpha_b$ is based on general practice where the stiffness component due to composite slab is ignored, and $\alpha_c$ for the column is derived from ACI 318.

Table C4: Stiffness Response Modification Factor

<table>
<thead>
<tr>
<th>Model Case</th>
<th>Joint Panel</th>
<th>Stiffness Modifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non Linear</td>
<td>Rot.Spring(Non Linear)</td>
<td>Per Eq(2-5) $I_g=\alpha_b x I_{go}$</td>
</tr>
<tr>
<td>Linear Model Case b</td>
<td>Rigid Connection B/W</td>
<td>$I_c=0.5xI_c0$ $I_g=\alpha_b x I_{go}$</td>
</tr>
<tr>
<td>Linear Model Case c</td>
<td>Neglecting joint size.</td>
<td>$I_c=0.7xI_c0$ $I_g=1.0xI_{go}$</td>
</tr>
</tbody>
</table>

$\alpha_b$: Average Modifier of Positive (as Composite Beam) and Negative Bending ($I_g; \alpha_b=1.3~1.5, (2I_g); \alpha_b=1.2~1.4$)

Results

Comparison of pushover analysis results of each Nonlinear Model, and the two linear cases are graphed in Figures C2 through C10 below.

Model **10 series

1. Model 0510 and Model 1010 show proper relation of stiffness with the nonlinear case.
2. Model 2010 shows overestimation of stiffness compared to the nonlinear case.
3. Case 1 exhibits higher stiffness than Case 2.

Model **05 series

1. Model 0505 and Model 1005 show proper relation of stiffness with the nonlinear case.
2. Model 2005 shows overestimation of stiffness compared to the nonlinear case.
3. Case 1 exhibits higher stiffness than Case 2.
Model **20 series

1. Model 0520 and Model 1020 show proper relation of stiffness with the nonlinear case.
2. Model 2020 shows overestimation of stiffness compared to the nonlinear case.
3. Case 1 exhibits higher stiffness than Case 2.

The trend is similar in each case and these assignments tend to overestimate the stiffness if beam is stiffer and stronger. Per our current design philosophies and regulations (strong column-weak beam), this case will never occur.
Figure C4: Model 2010

Figure C5: Model 0505
DESIGN OF COMPOSITE RCS SPECIAL MOMENT FRAMES

Model 1005

\[
\frac{I_g}{I_c} \quad \frac{L_g}{2L_c}
\]

**Figure C6: Model 1005**

Model 2005

\[
\frac{(2I_g)}{I_c} \quad \frac{L_g}{2L_c}
\]

**Figure C7: Model 2005**
Figure C8: Model 0520

Figure C9: Model 1020
Table C5 shows the initial stiffness and stiffness ratio of each Model series. The stiffness ratios of the linear model Case 1 are distributed from 0.97 to 1.17, and those of linear model Case 2 are distributed from 0.85 to 1.13.

Figures C11 and C12 show stiffness plotted for the Nonlinear Model and each Linear Model Case. Figure C12 shows that Case 2 stiffness plots are closer to the baseline and more reliable than Case 1.

<table>
<thead>
<tr>
<th>Model</th>
<th>Stiffness(kN/m)</th>
<th>Stiffness Ratio(Lin/Non Lin)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Non Lin</td>
<td>Case_1</td>
</tr>
<tr>
<td>0505</td>
<td>4492</td>
<td>4627</td>
</tr>
<tr>
<td>1005</td>
<td>3563</td>
<td>3805</td>
</tr>
<tr>
<td>2005</td>
<td>4697</td>
<td>5209</td>
</tr>
<tr>
<td>0510</td>
<td>21067</td>
<td>21882</td>
</tr>
<tr>
<td>1010</td>
<td>16255</td>
<td>17834</td>
</tr>
<tr>
<td>2010</td>
<td>22284</td>
<td>26003</td>
</tr>
<tr>
<td>0520</td>
<td>13346</td>
<td>12943</td>
</tr>
<tr>
<td>1020</td>
<td>11472</td>
<td>11502</td>
</tr>
<tr>
<td>2020</td>
<td>16598</td>
<td>17548</td>
</tr>
</tbody>
</table>
CONCLUSION

Linear Cases 1 and 2 exhibit similar stiffness behavior when compared to the nonlinear cases. Case 1 exhibits higher initial stiffness compared to Case 2. Case 2 also represents the current industry-accepted Stiffness Modification Factors. Therefore, use of Case 2 Stiffness Modification Factors, including beam stiffness modification factor $\alpha_b = 1.0$, and column stiffness modification factor $\alpha_c = 0.7$, may be used for appropriate linear modeling of Composite Frame Structures.
APPENDIX D
COST STUDY

Composite RCS is a structurally efficient high performing system that utilizes the inherent strengths of the steel and concrete materials. Earlier sections of this document discuss the design and construction of Composite RCS systems and compare their performance with steel moment frame construction. Does the improved performance come with additional cost? To explore the answer to the question, a sample cost study was performed using cost data from US, Japan and South-East Asian countries. The study is briefly disused in this Appendix and the results are presented.

The construction cost associated with the construction of a 54,000 square foot, three-story warehouse building using both steel moment frame system and Composite RCS frame system was developed and compared. These cost comparisons include only the frame components and do not include the floor deck system, the foundations or other parts of the structure that are only nominally affected by the selection of an RCS versus steel system.

BUILDING DESCRIPTION

Composite RCS Frame
The building described in design example in Section 2 of this document was used. See Section 2 for the building description and member sizes.

Steel Moment Frame System
The same building described in Section 2 was used, with the exception of steel columns in lieu of concrete columns.

Comparison of each structural system
Table D1 summarizes the member sizes for the Composite RCS, and Steel Moment Frame systems. Identical gravity and seismic loads were used for both structures and the members were designed to satisfy the requirements of ACI 318, AISC 360 and 341 and ASCE 7.
Table D1: Composite RCS and Steel Moment Frame Member Size Comparison

<table>
<thead>
<tr>
<th>Seismic Force Resistance System</th>
<th>Level</th>
<th>Column (Concrete)</th>
<th>Column (Steel)</th>
<th>Beam (Steel)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite RCS</td>
<td>3rd</td>
<td>32” x 32”</td>
<td>-</td>
<td>W21 x 62</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>32” x 32”</td>
<td>-</td>
<td>W30 x 108</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>32” x 32”</td>
<td>-</td>
<td>W30 x 108</td>
</tr>
<tr>
<td>Steel Moment Frame</td>
<td>3rd</td>
<td>-</td>
<td>W24x162</td>
<td>W21 x 62</td>
</tr>
<tr>
<td></td>
<td>2nd</td>
<td>-</td>
<td>W24x207</td>
<td>W30 x 108</td>
</tr>
<tr>
<td></td>
<td>1st</td>
<td>-</td>
<td>W24x207</td>
<td>W30 x 108</td>
</tr>
</tbody>
</table>

Material Specifications:
(a) Steel Frame: A992, $f_y = 50$ ksi
(b) High-strength bolts: A325 S.C.
(c) Welding electrodes: E70
(d) Concrete strength: $f_c' = 6,000$(psi)
(e) Rebar: Gr. 60

PARAMETERS

Construction costs from four different countries were developed and compared. United States, Japan, Thailand and Indonesia, which are the leaders in the use of Composite RCS technology, were selected. Steel and concrete are readily available in the United States and Japan, and the construction practices are advanced which incorporate lessons learned from previous earthquakes. The use of cast-in-place construction is prevalent in the developing South-East Asian countries, much more than pre-cast construction, due to material availability and the construction know-how.

Cast-in-place is assumed as the construction procedure for Composite RCS in Japan, Thailand and Indonesia due reasons discussed above. Pre-cast and cast-in-place concrete construction is considered for the United States.

FINDINGS

Table D2 Summary of Construction Cost, summarizes the estimated construction cost for each seismic system at each location considered. The cost is represented in US dollar per square foot.

The composite RCS frame is cost effective compared to the steel moment frame at all locations studied. This is primarily driven by the reduction in steel weight of the column and its replacement with slightly less expensive concrete. The labor cost associated with composite RCS is also smaller compared to the steel moment frame due to the simple fillet welded connections versus the complete joint penetration welds at commonly used moment frame joints.

Steel moment frame systems using fillet welds, may be competitive with the composite RCS, however the ease and familiarity with concrete construction in South-East Asian nations, may influence the final system selected.

The foundation systems and the floor deck are similar for both systems, therefore they were excluded from the study.
Table D2: Summary of Construction Cost\(^1\) (Composite RCS and Steel Moment Frame)

<table>
<thead>
<tr>
<th>Seismic Force Resistance System</th>
<th>Item</th>
<th>US (^2) CIP</th>
<th>US (^2) Pre-cast</th>
<th>Japan (^3) CIP</th>
<th>Thai (^4) CIP</th>
<th>Indonesia (^5) CIP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite RCS</td>
<td>Concrete</td>
<td>1.67</td>
<td>8.18</td>
<td>0.75</td>
<td>0.26</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>Re-bar</td>
<td>3.20</td>
<td></td>
<td>1.20</td>
<td>0.83</td>
<td>0.78</td>
</tr>
<tr>
<td></td>
<td>Form-Work</td>
<td>2.27</td>
<td></td>
<td>1.01</td>
<td>0.32</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>22.72</td>
<td>22.72</td>
<td>15.03</td>
<td>10.06</td>
<td>9.00</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>29.86</td>
<td>30.90</td>
<td>17.99</td>
<td>11.47</td>
<td>10.27</td>
</tr>
<tr>
<td>Steel Moment Frame</td>
<td>Concrete</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Re-bar</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Form-Work</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
<td>32.15</td>
<td>32.15</td>
<td>20.12</td>
<td>12.85</td>
<td>11.49</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td>32.15</td>
<td>32.15</td>
<td>20.12</td>
<td>12.85</td>
<td>11.49</td>
</tr>
</tbody>
</table>

1: All costs are in $/ft\(^2\)
2: Based on 2013 RS MEANS
3: Based on 105 Yen = 1 USD
4: Based on 328 Baht = 1 USD
5: Based on 11697.5 IDR = 1 USD

CONCLUSION

The Composite RCS systems demonstrate both structural performance and cost equivalence when compared to the Steel Moment Frame systems. The Composite RCS systems use the inherent strengths of steel and concrete materials and utilize them to the fullest and have the potential of being more cost effective than the conventional steel systems.
APPENDIX E
DRAFT - Pre-Standard for the Design of Moment Connections Between Steel Beams and Concrete Columns

PREFACE: This is an early draft of guidelines for the design of composite moment connections, which was originally drafted by the authors noted below. The guidelines are an update to an earlier set of guidelines that were originally published in 1994. The draft guidelines are included here as an interim resource for design guide for composite special moment frames.

Authors: Deierlein, G.G., Parra-Montesinos, G., Cordova, P., Bracci, J., Kanno, R.

Abstract: Equations and criteria are presented for the design of beam-column connections in composite moment frames consisting of structural steel or composite beams and reinforced concrete or composite columns. The proposed design model takes into account the interaction of the structural steel and reinforced concrete components in connections where the steel beam is continuous through the reinforced concrete column. This document constitutes a substantial update to guidelines first published in 1994 by an ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete. This update expands and refines the earlier guidelines based on new test data and research on reverse cyclic (earthquake) loading, alternative connection configurations, high strength concrete, and other design parameters, including provisions for seismic design that take advantage of the inherent connection strength and toughness. Commentary and a worked example are provided.

1. INTRODUCTION AND SCOPE

1.1 Application and Background

Criteria are presented herein for the design of beam to column moment resisting connections in composite RCS frames consisting of steel or composite beams and reinforced concrete or concrete encased composite columns. The provisions pertain to though-beam type connections, where the steel beam runs continuous through the column. This document represents a major update to design recommendations first published by an ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (ASCE 1994). The provisions include equations for calculating the strength and stiffness of the connection and requirements for proportioning structural steel and reinforcing bar details.
The term “composite RCS frame” refers to a moment resisting frame consisting of steel beams that may act compositely with a concrete slab and reinforced concrete or composite columns. Design and construction practice in the United States has been to apply RCS frames as the lateral force resisting system in moderate to high-rise buildings (Griffis 1986, 1992; Viest et al. 1997); whereas practice in Japan has been more geared toward seismic design applications in low- to moderate-rise buildings (Yamanouchi, et al. 1998). In mid- to high-rise applications, cast-in-place construction of the columns is common, whereas, in low-rise applications both cast-in-place and precast construction methods have been used. For cast-in-place construction, steel columns or sturdy reinforcing bar cages are used to support the steel framing prior to casting of the concrete columns. Where used, the embedded steel erection column is usually small relative to the moment strength of the steel beam and reinforced concrete column. Therefore, the connection design models neglect the direct moment transfer between the steel erection column and the steel beam.

These guidelines apply to beam-column connections for both cast-in-place and precast construction methods, however they are not intended to cover all types of connection configurations that are possible in composite systems. The guidelines are limited to “through-beam” type configurations where the steel beam runs continuous through the reinforced concrete column. A significant advantage of the through-beam configuration, particularly for seismic design, is to avoid interrupting the beam flanges at the point of maximum moment.

These design provisions are based largely on tests of through-beam planar beam-column joints, as shown in Fig. 1, subjected to forces associated with lateral deformation of the frame (Sheikh et al. 1987, 1989; Deierlein et al. 1988, 1989; Kanno 1993, Kanno and Deierlein 1996, 2000; Parra-Montesinos and Wight 2000a, 2000b; Parra-Montesinos et al. 2003; Liang et al. 2003, 2004). The guidelines have also been validated through a full-scale frame test (Chen et al. 2004; Cordova et al. 2004). Where applicable, the recommendations draw on existing provisions for steel and reinforced concrete joints, and other research on composite connections. The provisions are also informed by substantial research from Japan on RCS beam-column connections (Nishimura et al. 1986; Nishimura and Minami 1990; Nishiyama et al. 1990; Ogura et al. 1990; Yoshino et al. 1990, Noguchi and Uchida 2004; Nishiyama et al. 2004; Deierlein and Noguchi 2004).

1.2 Limitations

These provisions apply for the transfer of shear forces and moments in beam-column connections with the following limitations:
• Interior and exterior configurations in which the steel beam passes continuously through the reinforced concrete column. These provisions may be applied to top and corner configurations where the beam is anchored to the vertical column reinforcement.

• Joint aspect ratio: \( 2/3 \leq h/d \leq 3/2 \) where \( h \) = depth of concrete column measured parallel to the beam axis and \( d \) = depth of steel beam measured parallel to the column axis.

• Material specifications: normal-weight concrete with, for calculation purposes, \( f_{c'} \leq 100 \) MPa (15 ksi); reinforcing steel with \( F_y < 410 \) MPa (60 ksi); structural steel with \( F_y \leq 345 \) MPa (50 ksi).

In these provisions, the connection strength is determined by considering several individual failure modes, such as steel yielding or concrete crushing, as well as maximum allowable joint deformation. The guidelines are limited to conditions and configurations that are not substantially different from the tests upon which the design equations are based. The overall connection configurations are categorized as shown in Fig. 2. The accuracy of the overall design model in these provisions has been verified against tests of interior and exterior connections (Fig. 2a), built of concrete with normal-weight aggregate, and nominal concrete strengths up to \( f_{c'} = 100 \) MPa. Limited testing of top interior configurations indicates that these provisions can be adapted to the design of top interior and corner connections (Fig. 2b, provided that provisions are made to anchor the longitudinal column reinforcement at the top of the column (Cordova and Deierlein, 2005; other references?). Figure 3 shows an example of suggested techniques for anchoring the longitudinal column reinforcement. For more information on the supporting test data, the reader is referred to papers by cited in Section 1.1.

1.3 General Detailing Requirements

The connection detailing provisions include requirements for attachments to the structural steel beam, and transverse and longitudinal reinforcing bars in the joint region. Attachments to the steel beam include face bearing plates (FBPs), steel band plates, extended FBPs, steel cover plates, embedded steel columns, and vertical joint reinforcement. As a minimum, FBPs within the beam depth, equal in width to the beam flange, are required on all beams that frame into the column and are intended to transfer moment through the connection.

The details considered include the attachments to the steel beam shown in Figs. 1 and 4. These details can be used separately, or in combination with one another. FBPs are required for all connections since tests have demonstrated their effectiveness to significantly increase the joint strength and stiffness, and improved the joint performance by mobilizing the concrete
regions within the steel beam flanges and providing confinement to the concrete. The steel band plate detail (Fig. 1) is also quite effective in both mobilizing the outer concrete panel and confining concrete above and below the joint.

1.4 Constructability

The connection should be detailed to facilitate the anticipated construction sequence, including provisions for the erection of structural steel, and placement of reinforcing bars, installation of formwork, and casting of concrete.

The cost-effectiveness of composite systems is due in large part to efficient construction and erection procedures. These procedures can vary considerably depending on the project. Since composite construction involves the coordination of several trades, extra care should be used to ensure the constructability of the joint. To allow for proper casting of concrete in the joint, the steel beam flange width should be less than one-half of the column width (Deierlein et al. 1988). Further discussion of issues related to constructability is presented by Griffis (1986, 1992) Viest et al. (1997), Iyengar (1985), Moore and Gosain (1985), and Deierlein et al. (1988), Bracci et al. (1999).

2. DESIGN CONSIDERATIONS AND METHODOLOGY

2.1 Load and Resistance Factor Design

Wherever these provisions refer to the applicable building code and there is no local building code, the loads, load combinations, and general design requirements shall be those in ASCE 7-10 Minimum Design Loads for Buildings and Other Structures (2010). The design strength of the connection, which is equal to the product of the nominal connection strength and a resistance factor, shall be determined using these provisions.

These criteria for connection design are developed to be consistent with the general design and loading criteria of ASCE 7 (2010) and the associated member design criteria in the Specification for Structural Steel Buildings (ANSI/AISC 360, 2010), the Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341, 2010), and the Building Code Requirements for Structural Concrete and Commentary (ACI-318 2011). Since, by definition, composite RCS frames involve mixing of structural steel and concrete members, the frame and its components should be designed using load and resistance factors that are consistent between the various standards. This implies the use of the Load and Resistance Factor (LRFD) criteria in ANSI/AISC 360 and ACI-318, which are consistent with the ASCE-7 load combinations.
2.2 Connection Forces

These provisions are primarily intended for connections whose design is governed by the transfer of moments and shears between beams and columns through their common joint region. The connection should be designed for the factored load effects (bending, shear and axial forces) that are transmitted through the connected members.

In these provisions, the connection design forces are described by the applied forces calculated at the face of the connection, including column forces framing in from above \((Pc_1, Mc_1, Vc_1)\) and below \((Pc_2, Mc_2, Vc_2)\) the connection and beam forces framing in from the left \((Pb_1, Mb_1, Vb_1)\) and right \((Pb_2, Mb_2, Vb_2)\) of the connection. The required connection strength shall be calculated in terms of the resultant of these forces, as described through the following moment equilibrium equation,

\[
\Sigma M_c = \Sigma M_b + V_b h - V_c d
\]

where

\[
\Sigma M_b = (M_{b1} + M_{b2})
\]

\[
V_b = (V_{b1} + V_{b2})/2
\]

\[
V_c = (V_{c1} + V_{c2})/2
\]

\[
\Sigma M_c = (M_{c1} + M_{c2})
\]

and

\[
\Delta V_b = V_{b2} - V_{b1}
\]

\[
\Delta V_c = V_{c2} - V_{c1}
\]

For exterior, top, or corner connections, these same equations apply where the forces in the missing members are set to zero.

In structures designed to resist earthquakes through dissipation of energy through inelastic action, the required connection strength should be based on the expected strengths of the connected beams. For such cases, \(\Sigma M_b\) should be calculated considering material overstrength and strain hardening as specified in ANSI/AISC 341-10.

The forces acting on the connection are shown in Fig. 6, and the forces considered for the connection design in included in Eq. (1) to (7) are shown in Fig. 7. The connection design forces (Fig. 7) do not include the effect of the axial forces in the concrete column, and since axial forces in the beams are usually small, these are also excluded from the calculations. Test data on composite joints (Nishiyama et al. 1990; Minami 1985; Macrakis and Mitchell 1980; Kanno
1993) and design information on reinforced concrete joints (ACI-ASCE Committee 352 2002; Park and Paulay 1975) indicate that it is conservative to neglect the effects of axial compressive loads of magnitude less than that corresponding to the balance condition. This follows from tests where compressive axial stresses tended to inhibit the opening of cracks in the joint. The effects of axial tension in the column have not been tested and could result in a decrease of shear strength and stiffness in the joint. As described in Section 3, where large column tension forces exist, it is recommended that the outer concrete strut contribution to joint shear strength be neglected.

For design practice in the United States, composite frames have primarily been used in planar framing systems where floor beams framing in from the out-of-plane direction do not introduce significant joint forces. In general, the use of two-way (space) composite frames is complicated by the need to maintain beam continuity in the two orthogonal directions and by reinforcing bar congestion. Detailed description of acceptable two-way connection details can be found in Bracci et al. (1999) and Bugeja et al. (2000). Where space frames are used and beams frame into two orthogonal sides of the column, the available test data (Nishiyama et al 1998, 2000; Bugeja et al. 2000) indicates that it is reasonable to design the connection by independently applying the design guidelines for the maximum joint forces in each of the two orthogonal directions (i.e., it is permissible to neglect the interaction of biaxial joint loading effects on the connection).

2.3 Connection Strength

The connection strength should be checked for two basic failure modes: panel shear failure and vertical bearing failure. The strength equations given in Section 3 are based on these failure modes and depend on satisfying the detailing requirements in Section 4. The connection design strength is obtained by multiplying the nominal strengths of the inner and outer panels by a resistance factor, $\phi$, as specified in Section 3.

Connection behavior is characterized by the two modes of failure shown in Fig. 7. Panel shear failure (Fig. 7a) is similar to that typically associated with structural steel or reinforced concrete joints; however, in composite joints, both structural steel and reinforced concrete panel elements participate. Bearing failure (Fig. 7b) occurs at locations of high compressive stresses and may be associated with rigid body rotation of the steel beam within the concrete column. The vertical reinforcement shown in Fig. 4c and the steel band plates shown in Fig. 1 are methods for strengthening against bearing failure. Composite joint designs that may lead to premature bearing failures are not desirable. Test results (Kanno 1993, Kanno and Deierlein
have shown that joints failing in shear exhibit a superior displacement and energy dissipation capacity under reversed cyclic loading compared to those failing by bearing.

As specified later in Section 3, for connections whose strength is controlled by joint shear $\phi_s = 0.85$; and for connections whose strength is controlled by joint bearing $\phi_b = 0.75$. The value of $\phi_s = 0.85$ is comparable to the resistance factor applied in the design of composite columns in ANSI/AISC-360. Comparisons with available test data for connections failing in shear indicate that the resulting design strength ($\phi_s R_{ns}$) provides a reliability index of $\beta = 3.0$ (Cordova et al., 2006). A lower $\phi$ factor of $\phi_b = 0.75$ is used in joint designs controlled by bearing since data indicates that connections failing in bearing exhibit less ductility than those failing in shear (Kanno 1993). Less data is available regarding joint bearing failure, though the available data indicated that the reliability index for the design joint bearing strength ($\phi_b R_{nb}$) is about $\beta = 4.5$ (Cordova and Deierlein 2005).

### 2.4 Connection Stiffness

Connection deformations should be considered in evaluating frame deflections under service and strength limit states. Joints designed according to these provisions are expected to experience a total angular connection distortion of 0.006 radian at applied loads equal to one-half of the nominal connection strength and 0.02 radian at applied loads equal to the nominal strength.

Available test data indicate that the joint response may be idealized by the plot in Fig. 8 relating the applied joint shear to the total joint distortion. In Fig. 8, the joint shear, $V_j$, has been normalized by the nominal joint strength, $V_n$. The idealized joint behavior illustrated in Fig. 8 has been calibrated with results from tests of several interior and exterior connections failing in shear and bearing (Kanno 1993; Parra-Montesinos and Wight 2000; Parra-Montesinos et al. 2003; Liang et al. 2003) and may be used to simulate the response of RCS connections.

### 3. NOMINAL STRENGTH

#### 3.1 Effective Joint Width

The effective width of the joint ($b_j$) within the column is equal to the sum of the inner and outer joint panel widths ($b_i$ and $b_o$),

$$b_j = b_i + b_o$$ (8)

The inner width $b_i$ should be taken equal to the beam flange width, $b_f$. The outer panel width $b_o$ is calculated based on the connection geometry and the shear keys (such as steel columns,
steel band plates, extended FBPs) used to mobilize the concrete regions outside the width of the steel beam flanges. Where shear keys are provided in the form of extended FBPs, band plates, or steel columns, the outer panel width is determined as,

$$b_o = y + 2/3 \alpha x - b_f \leq b - b_f$$

(9)

where, $b =$ the concrete column width measured perpendicular to the beam, $y =$ the steel column or extended FBP width or $y = b_f$ for the band plate. For extended FBPs and band plates, $x = h$, and for encased steel columns, $x = h/2 + d_c/2$ where $d_c =$ the steel column depth. When more than one shear key exists, the outer panel width should be determined based on the shear key that provides the largest width. If no shear keys are present, the outer panel width may be calculated using $x = 0.7h$ and $y = 0$, based on friction resistance between the beam and concrete in the vertical bearing region. Where steel band plates are used in lieu of reinforcing bar ties in the bearing region above and below the beam, $\alpha = 0.5$; otherwise, where ties are present (with or without the band plate), $\alpha = 1.0$.

The joint shear strength is calculated based on an effective joint width, which is the sum of the inner and outer panel widths, as shown in Fig. 9. The concrete in the inner panel is mobilized through bearing against the FBPs between the beam flanges. The participation of concrete outside of the beam flanges for connections with the shear keys is dependent on mobilization of the horizontal compression struts that form through direct bearing of the shear keys on the concrete above and below the joint, as shown in Fig. 10. Where extended FBPs, band plates, or steel columns are used, the outer panel width is roughly determined by projecting a line with a 1:3 slope from the edges of the shear keys to the edge of the column. The outward thrust at the end of the compression struts is resisted by horizontal ties (or the band plates) above and below the beam. Referring to Fig. 10, the ties (or band plates) above and below the beam are required to resist tension forces both parallel and perpendicular to the beam. The forces perpendicular to the beam are self-equilibrating and those parallel to the beam are transferred into the outer concrete strut. For connections with steel band plates, a direct activation of the outer concrete strut through bearing against the plates perpendicular to the beam is assumed. The minimum outer panel width corresponding to $x = 0.7h$ and $y = 0$ is determined as the distance to the approximate centroid of the bearing region. The effective joint width is used in Section 3.4 to calculate joint strength. The lower $\alpha$ factor($=0.5$) for band plates without interior ties accounts for the fact that the exterior band along does not provide as direct a load path to mobilize the entire outer joint panel region.
3.2 Joint Strength

The design shear strength of the joint is the sum of the nominal resistance of the inner panel $V_{in}$, which includes the contribution from the steel web panel and an inner concrete strut, and the outer concrete panel, $V_{on}$, multiplied by their respective resistance factors. The inner and outer panel strengths, $V_{in}$ and $V_{on}$, should be determined following the procedures given in Sections 3.2.1 and 3.2.2, respectively. The shear strength of the connection is considered adequate if the following equation is satisfied:

$$V_j \leq k (\phi V_{in} + \phi V_{on})$$

(10)

where, $k$ is an adjustment factor to control connection deformations ($k = 1.0$ where large connection deformations are considered in the design, and $k = 0.85$ where the intent is to limit joint deformations), $\phi_s = 0.85$ for shear in the outer panel, and $\phi$ is specified depending on the governing design check of the inner panel in Section 3.4.1. The required connection strength, $V_j$, in (10) is calculated as,

$$V_j = \sum \frac{M_b}{d_j} - V_c$$

(11)

where $\sum M_b$ and $V_c$ are as defined in Section 2.2, and $d_j$ is the effective joint depth, determined as the distance between steel beam flange centerlines.

The joint shear mechanisms are shown in Fig. 11. Tests have shown that the contributions of the mechanisms are additive. The concrete contribution comes from the concrete compression struts that form within the inner panel width, $b_i$, and the outer panel width, $b_o$ (Fig. 9). The inner concrete compression strut (Fig. 11b) is activated through bearing against the FBPs and steel beam flanges. The outer concrete compression strut (Fig. 11c) is mobilized either through a horizontal strut and tie mechanism that forms through bearing against the steel column and/or extended FBPs, or through direct bearing against steel band plates (see Fig. 10).

Equation (10) is derived by equating the horizontal shear through the joint due to the applied loads to the total design joint shear strength. The design joint shear strength, the right side of (10), is the sum of the horizontal shear strength of the inner and outer panels, as shown in Fig. 11, multiplied by a strength reduction factor $k$ intended to control joint distortions such that connection damage is limited to moderate cracking with web panel yielding under factored or ultimate loads. Where the joint is sized to develop the full strength of the beams and some joint flexibility is accounted for in analysis (e.g., by using centerline frame dimensions), the connection deformation factor would usually be take as $k = 1.0$. Eq. (11) is the applied...
horizontal joint shear in terms of the total applied beam moments, \( \Sigma M_b \), and the column shear, \( V_c \).

### 3.2.1 Inner Panel

The design strength of the inner panel, \( V_{in} \), is governed by either the combined horizontal shear strengths of the steel web panel, \( V_{spn} \), and inner diagonal concrete strut, \( V_{icn} \), or by the vertical bearing capacity of the inner panel. Thus,

\[
\phi V_{in} = \phi_s \left( V_{spn} + V_{icn} \right) \leq \phi_b \left( \frac{M_{vb} - V_b h}{d_j} \right)
\]

where \( V_{spn}, V_{icn}, \) and \( M_{vb} \) are determined by Eq. (13) – (15), \( \phi_s = 0.85 \), and \( \phi_b = 0.75 \).

The horizontal shear strength of the steel web panel and the inner diagonal concrete strut are determined as,

\[
V_{sp} = 0.6 F_{ysp} t_{sp} \alpha_{sp} h
\]

\[
V_{ic} = 1.7 \alpha_c \sqrt{f_c' b} h \leq 0.5 f_c' b_{ij} d_j
\]

\( F_{ysp} \) and \( t_{sp} \) are the yield strength and thickness of the steel panel, respectively, \( \alpha_{sp} = 0.9 \) and 0.8 for interior and exterior connections, respectively, \( \alpha_c = 1.0 \) and 0.6 for interior and exterior connections, and \( f_c' \) is the concrete compressive strength in MPa.

The moment bearing strength of the connection, \( M_{vb} \), is determined as,

\[
M_{vb} = C_{cn} h \left( 1 - \beta_1/2 \right) + h_{vr} (T_{vrn} + C_{vrn})
\]

\[
C_{cn} = \alpha_{cn} f_{c}' b_{ij} \beta_1 h/2
\]

where, \( T_{vrn} \) and \( C_{vrn} \) = the nominal strengths in tension and compression, respectively, of the vertical joint reinforcement, which is attached directly to the steel beam, and \( h_{vr} \) = the distance between the bars. For concrete strengths up to and including 27.6 MPa (4 ksi) the factor \( \beta_1 = 0.85 \), and for strengths greater than 55.2MPa (8 ksi) \( \beta_1 = 0.65 \). For concrete strengths between these limits, \( \beta_1 \) should be linearly interpolated. For bearing regions with minimum reinforcing bar ties, \( \alpha_{cn} = 2.0 \); and for bearing regions confined by band plates, \( \alpha_{cn} = 2.5 \).

The following factors should be considered in the strength calculation for \( T_{vrn} \) and \( C_{vrn} \): connection between the reinforcement and steel beam, development of the reinforcement through bond or anchorage to concrete, and the material strength of the reinforcement. In addition, for use in (15), the contribution of the vertical reinforcement is limited as follows:
\[ T_{\text{vrn}} + C_{\text{vrn}} \leq \frac{C_{\text{cn}}}{2} \]  

The strength of the inner panel is governed by either the shear strength of the steel web panel and inner concrete strut, or by the bearing resistance of the inner panel. The right-hand side of Eq. (12) is the horizontal joint shear that corresponds to the bearing resistance of the inner panel.

The nominal horizontal shear strength of the steel web panel, \( V_{sp} \), is calculated based on an average shear yield strength of \( 0.6F_{\text{ysp}} \), acting over an effective joint length, \( \alpha h \). Results from research (Parra-Montesinos and Wight 2000; Noguchi and Kim 1998) have shown that a shorter steel panel length is mobilized in exterior joints compared to interior joints.

The strength of the inner diagonal compression strut was determined based on semi-empirical models developed for RCS connections considering the beneficial effect of confinement by the steel flanges and FBPs in the inner concrete panel, and softening of concrete due to diagonal cracking (Parra-Montesinos and Wight 2001). The coefficient of 1.7 is equal to the strength factor given in ACI-ASCE 352-R02 (2002) for monolithic reinforced concrete joints confined on all four faces. To prevent bearing (crushing) failure at the ends of the strut, the horizontal shear in (12) is limited by a bearing stress of \( 2.5f'_{c} \) over an area at the top and bottom of the FBPs equal to \( 0.20b_{f}d_{j} \).

The vertical bearing forces on the joint are due to the combined effects of moments and shears transferred between the beam and column. The moments and shears acting on the joint are shown in Fig. 12b. In Fig. 12c, the column moments, \( M_{c1} \) and \( M_{c2} \), are replaced with the forces in the vertical reinforcement, \( T_{\text{vr}} \) and \( C_{\text{vr}} \), the vertical bearing forces, \( C_{c} \), and the vertical forces acting in the outer concrete strut. The beam shears in Figs. 12b and 12c are related to (2)-(7). The lengths of the bearing zones above and below the beam, \( a_{c} \), are assumed to be equal to \( \beta_{1}h/2 \). This limit is based on test data (Kanno 1993) and is used in lieu of a limitation on the maximum concrete strain. The nominal vertical concrete bearing strength, \( C_{\text{cn}} \), is calculated using a bearing stress of \( 2.0f'_{c} \) over a bearing area with length, \( a_{c} = \beta_{1}h/2 \), and width equal to the width of the steel beam flange, \( b_{f} \). The maximum bearing stress of \( 2.0f'_{c} \) reflects confinement of the concrete by steel hoops. For connections with steel band plates, the maximum bearing stress is increased to \( 2.5f'_{c} \). These values are based on test data from joint tests (Sheikh et al. 1987, 1989; Deierlein et al., 1988; Kanno 1993; Kanno and Deierlein 2000; Parra-Montesinos and Wight 2000; Liang and Parra-Montesinos 2004) and direct bearing tests (Minami 1985; Kanno 1993) and is comparable to the maximum bearing stress allowed beneath an anchorage plate (2.25\( f'_{c} \)) in Caltrans-AASHTO (2000).
Vertical joint reinforcement may consist of reinforcing bars, rods, steel angles, or other elements attached directly to the steel beam to transfer vertical forces into the concrete column. Depending on the type of connection to the steel beam, the reinforcement may be considered to act in both tension and compression or in compression only ($T_{vr} = 0$). Vertical stiffeners or other details may be required to transfer the forces in the vertical reinforcement into the web of the steel beam. If the amount of vertical reinforcement is too high, there is a concern that the joint concrete between the top and bottom flanges of the steel beams may be subjected to excessive bearing stresses. Equation (18) provides an upper limit on the contribution of vertical joint reinforcement to the joint bearing capacity.

3.4.2 Outer Concrete Strut

The nominal strength of the outer diagonal concrete strut, $V_{on}$, shall be determined as:

$$V_{on} = 1.25\alpha_c \sqrt{f_c b_o h}$$

(18)

where $\alpha_c$ is as defined previously for the inner panel (14) and the other terms are as defined previously. Where the column is subjected to a net tension force larger than $0.1f'cA_g$ under the design loads, then $V_{on} = 0$. Hoops within the beam depth as well as adjacent to the joint, and steel band plates should satisfy the requirements given in Section 4.1.

The equations for calculating the outer concrete strut strength are similar to those used for the inner concrete strut. The strength assigned to the inner strut, however, is larger because of the more favorable confinement conditions in the inner concrete regions of the connection provided by the steel beam flanges and FBPs.

4. DETAILING CONSIDERATIONS

4.1 Confinement requirements within the Beam Depth

Concrete within the beam depth shall be confined by horizontal reinforcing bars or other means, such as steel cover plates. Where horizontal reinforcing bar ties are used, they should be provided in the column within the beam depth with a minimum volumetric ratio of $\rho_s = 0.01$, where $\rho_s$ is the ratio between the tie volume within the beam depth and the joint volume ($d_j/h_b$). The maximum spacing between horizontal ties should not exceed the smaller of $0.25d_j$ and $0.25h$. Perimeter ties and cross ties may be developed by either 90° hooks, which engage a longitudinal bar, or by lap splicing the ties. For connections with beams framing in two orthogonal directions, the steel beams with FBPs may be considered as providing confinement.
within the joint depth. In such cases, steel ties, anchored in the inner concrete panel, should be used to provide support to the longitudinal column bars. Hook details and splice lengths should conform to the provisions of ACI 318-11.

As illustrated in Fig. 13, where the column is large and members frame in from four sides, it is possible to use isolated hoops within the joint that are anchored within the confined concrete regions behind the FBPs.

4.2 Concrete Confinement Adjacent to Connection

Concrete confinement in the column above and below the beam shall be provided by reinforcing bar ties or steel band plates. Where band plates are used, the band should extend at least $0.25d_j$ above and below the beam and should be attached to the beam flanges and stiffened with a vertical stiffener plate. The steel band plates should have a minimum volume of $0.03 \frac{d_j}{h} b$ and a minimum thickness of 13 mm. For joints without band plates, a minimum of three layers of ties should be provided above and below the beam, and the bars in each layer should be at least equivalent to the following: for $b \leq 500$ mm, four 10-mm bars; for $500$ mm < $b \leq 750$ mm, four 13-mm bars; and for $b > 750$ mm, four 16-mm bars. These ties should be closed rectangular ties that can resist tension parallel and perpendicular to the beam. For seismic design the ties should satisfy seismic hoop requirements of ACI 318-05. The three layers should be located within a distance of $0.4d_j$ above and below the beam.

When band plates, extended FBPs, or embedded steel columns are used to activate the outer concrete strut, the reinforcing bar ties or band plate above and below the beam may be governed by the need to transfer the force $V_{on}$ from the beam flanges into the outer concrete panel. The minimum total cross-sectional area $A$ of the ties or band plate should meet the following

$$A \geq \frac{V_{on}}{F_y}$$

where $V_{on}$ = the nominal horizontal shear strength of the outer concrete strut, $F_y$ = the yield strength of the reinforcement or band plate. The calculated area, $A$, is the total cross-sectional area (measured through a vertical plane perpendicular to the beam) of the band plate or ties located within a vertical distance $0.4d_j$ above and below the steel beam.

Within the beam depth, horizontal ties confine the outer diagonal concrete strut (Fig. 10c). Ties above and below the beam also participate in the horizontal strut and tie mechanism, which transfers the shear outwards to the outer concrete compression strut. Ties also provide confinement of the concrete in and adjacent to the joint. The provisions in Section 4.1 are based

4.3 Longitudinal Column Reinforcing Bars

The size of vertical column bars passing through the joint should be limited as follows:

\[ d_b \leq \frac{d_j \times (420)}{20 \times F_{yr}} \leq \frac{d_j}{20} \]  

(20)

where, for single bars, \( d_b \) = vertical bar diameter, and for bundled bars, \( d_b \) = the diameter of a bar of equivalent area to the bundle, and \( F_{yr} \) = yield strength of longitudinal column bars in MPa. When steel band plates are used, the joint depth may be assumed equal to \( 1.25 \times d_j \).

The limit on bar size [(20)] is based on similar limits proposed for reinforced concrete joints to limit bar slip associated with possible large changes in bar forces due to the transfer of moments through the joint (ACI-ASCE 2002). Results from tests of RCS connections under load reversals have shown that, although slip of column longitudinal bars is not fully eliminated, it is limited to acceptable levels when (20) is satisfied (Parra-Montesinos and Wight 2000b).

4.4 Steel Beam Flanges

The vertical bearing force associated with joint shear in the steel panel causes bending of the steel beam flanges. The beam flanges can be assumed capable of resisting transverse bending if the thickness satisfies the following:

\[ t_j \geq 0.30 \sqrt{\frac{b_j \times t_{sp} \times d \times F_{ypp}}{h \times F_{yf}}} \]

(21)

where \( t_{sp} \) and \( F_{ypp} \) = the thickness and yield strength of the steel panel and \( F_{yf} \) = the yield strength of the beam flanges.

Equation (21) is a semiempirical formula derived from joint tests (Sheikh et al. 1987) for a bearing force equal to the shear strength of the steel panel. If the thickness of the beam flange does not satisfy (21), the flange should be reinforced to increase its transverse bending strength to carry a bearing force equal to the vertical shear strength of the steel panel. Reinforcement could consist of additional vertical stiffeners or horizontal bearing plates welded to the flanges.
4.5 Face Bearing Plates within Beam Depth

The FBPs within the beam depth should be detailed to resist a horizontal shear force equal to the horizontal nominal shear strength of the inner concrete strut, $V_{icn}$. The FBP thickness, $t_p$, should meet the following conditions,

$$t_p \geq \frac{\sqrt{3} V_{icn}}{2b_f F_{up}}$$  \hspace{1cm} (22)

$$t_p \geq 0.20 \left[ \frac{V_{icn} b_f}{F_{yp} d_w} \right]$$  \hspace{1cm} (23)

where $F_{yp}$ and $F_{up}$ = the specified yield strength and tensile strength of the bearing plate, respectively.

Also, the thickness $t_p$ should be such that

$$t_p \geq b_f / 22$$  \hspace{1cm} (24)

The required thickness of the FBPs is a function of their geometry, support conditions, yield strength, and the distribution of concrete bearing force. Since the distribution of bearing force is nonuniform, traditional methods of analysis (e.g., yield line method) are not appropriate and usually result in overly conservative thickness. Equations (22) and (23) are semiempirical formulas derived from joint tests (Sheikh et al. 1987). Equation (22) limits shear stresses in the FBPs, while (23) limits flexural bending stresses. Welds connecting the plates to the beam should be proportioned for the full capacity of the plate both in shear and flexure.

4.6 Bearing Resistance of Steel Band Plates, Extended Face Bearing Plates and Steel Column

Where used, the band plates, extended FBPs, and/or steel column should be designed to resist a force equal to the joint shear carried by the outer compression strut, $V_{on}$. The average concrete bearing stress against these elements should be less than or equal to $2f'_c$ and should be considered to act over a height not exceeding $0.25d_j$.

The minimum thickness of the column flanges, band plate, or extended FBPs shall satisfy,

$$t \geq 0.12 \left[ \frac{V_{on} b_p}{d_{hp} F_{yp}} \right]$$  \hspace{1cm} (25)
\[ t \geq \frac{\sqrt{3}}{2 \beta_p F_{ubp}} V_{on} \]  

(26)

where \( d_{bp} \) = depth of steel band plate \((0.25d_i)\); \( F_{ybp} \) and \( F_{ubp} \) = the specified yield strength and tensile strength of the steel band plates, respectively.

The extended FBPs and/or the steel columns are required to bear against the horizontal compression struts, as shown in Fig. 9. The maximum net bearing force parallel to the beam is equal to the horizontal shear force carried in the outer concrete strut mechanism, \( V_{on} \). As shown in Fig. 9, when a steel column is used, most of the force is transferred through bearing to only one of the column flanges. The design of these elements is usually controlled by transverse bending in the plates or column flanges, shear strength of the support plate or the column web, and the connection to the steel beam. The maximum effective height of the bearing region is assumed equal to \( 0.25 \, d_i \), based on the limits of the available test data.

As mentioned in the commentary for Section 4.5, traditional methods of analysis for the flexural bending of the extended FBP (or column flanges) usually result in overly conservative thicknesses. Equation (25) is a semiempirical formula derived from joint tests (Deierlein et al. 1988) and is based on flexural bending considerations only. The extended FBP or steel column flanges should also be checked against shear fracture. Welds connecting these plates to the steel beam should be proportioned for the full capacity of these plates both in shear and flexure.
APPENDIX I. REFERENCES


ANSI/ASCE 7 (2010), Minimum design loads for buildings and other structures.” ASCE, http://dx.doi.org/10.1061/9780784412916


APPENDIX II. NOTATION

The following symbols are used in this paper:

\( A_{sh} \) = cross-sectional area of reinforcement parallel to beam (including cross ties) with spacing \( s_h \);

\( A_{tie} \) = total area of ties parallel to beam and located within 0.4d of beam;

\( a_c \) = length of concrete bearing zone;

\( b \) = width of concrete column measured perpendicular to beam;

\( b_f \) = width of steel beam flanges;

\( b_i \) = width of inner concrete panel;

\( b_j \) = effective width of joint panel;

\( b_o \) = effective width of outer concrete panel;

\( b_p \) = width of FBP;

\( b_{p} \) = width of steel column or extended FBP;

\( C_c \) = compression force in concrete bearing zone;

\( C_{cn} \) = nominal compression strength of bearing zone;

\( C_{vr} \) = compression force in vertical reinforcement;

\( C_{vrn} \) = nominal compression strength of vertical reinforcement;

\( D \) = depth of steel beam measured parallel to column;

\( d_b \) = diameter of reinforcing bar;

\( d_{bp} \) = depth of steel band plate measured parallel to column;

\( d_c \) = steel column depth measured parallel to beam;

\( d_j \) = effective joint depth;

\( d_o \) = additional effective joint depth provided by attachments to beam flanges;

\( d_w \) = distance between beam flanges (height of web);

\( F_{1,2} \) = theoretical forces in vertical column bars above and below joint;

\( F_{up} \) = specified tensile strength of bearing plate;

\( F_{ubp} \) = specified tensile strength of steel band plate;
\( F_{yf} = \) specified yield strength of beam flange; \\
\( F_{yp} = \) specified yield strength of bearing plate; \\
\( F_{ysp} = \) specified yield strength of steel joint panel \\
\( F_{ysh} = \) specified yield strength of column ties; \\
\( F_{yw} = \) specified yield strength of beam web; \\
\( f_b = \) vertical bearing stress \\
\( f'_c = \) specified compression strength of concrete; \\
\( h = \) depth of concrete column measured parallel to beam; \\
\( h_{vr} = \) horizontal distance between vertical joint reinforcement; \\
\( k = \) deformation-based strength reduction factor; \\
\( M_{b1,2} = \) moments in steel beams adjacent to joint; \\
\( M_{c1,2} = \) moments in concrete columns adjacent to joint; \\
\( M_{vb} = \) bearing moment strength of joint; \\
\( P_{b1,2} = \) axial forces in steel beams adjacent to joint; \\
\( P_{c1,2} = \) axial forces in concrete columns adjacent to joint; \\
\( P_n = \) nominal compression strength of column cross section; \\
\( P_u = \) factored axial compression load in column; \\
\( s_h = \) center-to-center spacing of column ties; \\
\( T_{vr} = \) tension force in vertical joint reinforcement; \\
\( T_{vrm} = \) nominal tension strength of vertical joint reinforcement; \\
\( t_f = \) thickness of beam flanges; \\
\( t_p = \) thickness of FBPs; \\
\( t_{sp} = \) thickness of steel joint panel; \\
\( t_w = \) thickness of beam web; \\
\( V_b = \) antisymmetric portion of beam shears; \\
\( V_{b1,2} = \) shears in steel beams adjacent to joint; \\
\( V_c = \) antisymmetric portion of column shears;
$V_{c1,2}$ = shears in concrete columns adjacent to joint;

$V_{cin}$ = horizontal shear strength of concrete strut

$V_{on}$ = nominal horizontal shear strength of outer concrete strut;

$V_j$ = total horizontal joint shear force;

$V_{snp}$ = nominal horizontal shear force in steel panel;

$x$ = dimension for calculation of effective joint width (Fig. 9);

$y$ = dimension for calculation of effective joint width (Fig. 9);

$\alpha h$ = effective joint length in direction of beam

$\phi$ = resistance (capacity reduction) factor;

$\Delta V_b$ = net vertical beam shear transferred into column;

$\Delta V_c$ = net horizontal column shear transferred into beam;

$\Sigma M_c$ = net column moments transferred through joint; and

$\Sigma M_b$ = net beam moments transferred through joint.
FIGURES

Figure 1 – Overall configuration of through-beam RCS connection (Cordova & Deierlein, 2005)

Figure 2 – Joint configurations (ASCE 1994)
Figure 3 – Cap detail to anchor longitudinal reinforcement at top-interior joint (Cordova and Deierlein 2005).
Figure 4 – Alternative joint details (a) FBP, (b) Extended FBP and Steel Column, and (c) Vertical Joint Reinforcement (ASCE 1994). Also see Band Plate detail in Figure 1.
Figure 5 – Member forces acting on joint (ASCE 1994)

Figure 6 – Joint design forces: (a) Interior, (b) Exterior, (c) Top-Interior, and (d) Top-Exterior (adapted from ASCE 1994)
Figure 7 – Typical joint failure modes (Kanno et al. 2000)

Figure 8 – Idealized joint shear force versus joint shear deformation (Parra-Montesinos et al. 2001)
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Figure 11 – Calculation of joint shear strength (a) steel web panel, (b) concrete compression strut, (c) concrete compression field (ASCE 1994)
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Figure 13 – Alternative detail of confinement reinforcement within the joint (Cordova & Deierlein 2005)