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Design of RBS Connections for Special Moment Frames

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This publication provides guidance into the design of reduced beam section (RBS) steel special moment frame (SMF) connections. Many connection types are available for use in the design of steel moment resisting frames, but only a few connections are established as robust options for maintaining vertical load carrying capacity when subjected to large rotational demands. The RBS is currently the only nonproprietary connection type preapproved for use in SMF structures.

The American Institute of Steel Construction (AISC) has recently released *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358, 2005), a national standard for the design of the RBS connection type. This document provides guidance to the engineer in the design of this connection, including the consideration of bracing different conditions required by the AISC's *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, 2005).

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Notations

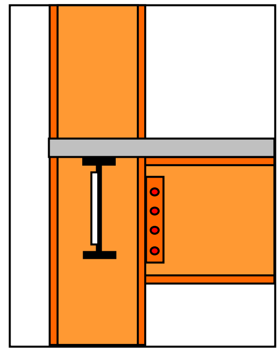
The standard uses the following symbols in addition to some of the standard terms and symbols defined in the AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, 2005) and the AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, 2005).

A_g	Gross cross-sectional area of member, in.^{2} (mm^2)
A_{pb}	Contact areas between the continuity plate and the column flanges that have attached beam flanges, in.^{2} (mm^2)
A_w	Areas of beam web, in.^{2} (mm^2)
A_{pw}	Contact areas between the continuity plate and the column web, in.^{2} (mm^2)
C_{bb}	Moment diagram modification factor for full bracing condition
C_d	Buckling shape factor
C_{pr}	Factor to account for peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection conditions, as given in Equation 2.4.3-2 (ANSI/AISC 358, 2005)
C_v	Web shear coefficient
DL	Dead load
D_{min}	Minimum required fillet weld size, in. (mm)
E	Elastic modulus, ksi (MPa)
Eqn	Equation
F_{br}	Brace force for calculation of brace stiffness, kips (N)
F_f	Flange force (from the force couple developed from M_f), kips (N)
F_{fu}	Ultimate flange force, kips (N)
F_{su}	Ultimate stiffener force, kips (N)
F_u	Ultimate yield stress of the steel section, ksi (MPa)
F_v	Nominal shear strength of bolts, ksi (MPa)
F_y	Specified minimum yield stress of the steel section, ksi (MPa)
F_{yb}	Specified minimum yield stress of the beam flange, ksi (MPa)
F_{yc}	Specified minimum yield stress of the column flange, ksi (MPa)
I_{brace}	Brace element moment of inertia for bracing calculations, in.^{4} (mm^4)
I_{eff}	Effective moment of inertia for bracing calculations, in. (mm)
I_{yc}	Lateral moment of inertia for compression flange, in. (mm)
I_{yt}	Lateral moment of inertia for tension flange, in. (mm)
L	Distance between the centers of adjacent columns along a moment beam, in. (mm)
LL	Live load
L'	Distance between the centers of the reduced beam section, in. (mm)
L_b	Unbraced length of the moment beam, in. (mm)
L_{br}	Maximum brace spacing distance, in. (mm)
L_o	Length of beam between the centerline of the columns, in. (mm)
L_{brace}	Length of brace, in. (mm)
L_{pbm}	Length of perpendicular bracing beam, in. (mm)
M_{cf}	Column flange flexural strength, kip-in. (N-mm)
M_f	Maximum moment expected at face of column, kip-in. (N-mm)
$M_{gravity}$	Beam moment resulting from $1.2DL + f_1LL + 0.2SL$, kip-in. (N-mm)
M_{gvity}	Beam moment resulting from $1.2DL + f_1LL + 0.2SL$, kip-in. (N-mm)
M_n	Nominal moment strength, kip-in. (N-mm)
M_{pc}	Plastic moment of column based on expected yield stress, kip-in. (N-mm)
M_{pe}	Plastic moment of beam based on expected yield stress, kip-in. (N-mm)
M_r	Required flexural strength, kip-in. (N-mm)

M_{pr}	Probable maximum moment at plastic hinge, kip-in. (N-mm)
M_u	Maximum moment in beam (for bracing), kip-in. (N-mm)
M_v	Moment in beam because of vertical loads, kip-in. (N-mm)
P_{br}	Beam bracing force, kip (N)
P_{uc}	Ultimate axial force in column, kip (N)
R	Radius of reduced beam section (RBS) cut, in. (mm)
$R_{ct\text{-}cf}$	Required strength of the continuity plate at the projected bearing surface (column flange), kips (N)
$R_{ct\text{-}cw}$	Required strength of the continuity plate at the projected bearing surface (column web), kips (N)
R_n	Required force for stiffener design, kips (N)
R_u	Panel zone design shear, kips (N)
R_v	Panel zone shear strength, kips (N)
R_y	Ratio of expected yield stress to specified minimum yield stress F_y, as specified in the AISC *Seismic Provisions* (ANSI/AISC 341, 2005)
R_{yb}	Ratio of expected yield stress to specified minimum yield stress F_y, for a beam
R_{yc}	Ratio of expected yield stress to specified minimum yield stress F_y, for a column
S_h	Distance from the center of a column to the center of plastic hinge, in. (mm)
S_{xbrace}	Section modulus of bracing beam, in.^{3} (mm^3)
V_D	Beam shear force at the center of the RBS caused by dead load, kips (N)
V_L	Beam shear force at the center of the RBS caused by live load, kips (N)
$V_{gravity}$	Beam shear force resulting from $1.2DL + f_1LL + 0.2SL$, kips (N)
V_{nw}	Design shear force at the continuity plate to column web interface, kips (N)
V_n	Nominal shear strength, kips (N)
V_p	Beam shear force at the center of the RBS section, kips (N)
V_{pr}	Beam shear force at the center of the RBS caused by plastic moment capacity of the beam at the center of the RBS, kips (N)
V_{RBS}	Larger of the two values of shear force at the center of the reduced beam section at each end of a beam, kips (N)
V'_{RBS}	Smaller of the two values of shear force at the center of the reduced beam section at each end of a beam, kips (N)
V_u	Required shear strength of beam and beam web-to-column connection, kips (N)
V_p	Shear force at the center of the RBS, kips (N)
$W_{pb\text{-}flange}$	Width of the continuity plate in contact with the column flanges, in.^{2} (mm^2)
Z_x	Plastic section modulus of a member, in.^{3} (mm^3)
Z_b, Z_{xb}	Plastic section modulus of a beam, in.^{3} (mm^3)
Z_{xc}	Plastic section modulus of a column, in.^{3} (mm^3)
Z_e	Effective plastic modulus of a section (or connection) at the location of a plastic hinge, in.^{3} (mm^3)
Z_{RBS}	Plastic section modulus at the center of the reduced beam section, in.^{3} (mm^3)
a	Horizontal distance between a column flange and the start of an RBS cut, in. (mm)
b	Length of an RBS cut, in. (mm)
b_{bf}	Width of beam flange, in. (mm)
b_{cf}	Width of column flange, in. (mm)
$b_{cont\text{-}pl}$	Width of the continuity plate (face of column web column to edge of column flanges), in.^{2} (mm^2)
b_{stiff}	Width of stiffener plate, in. (mm)
c	Depth of cut at the center of the reduced beam section, in. (mm)
d_b	Beam depth, in. (mm)
d_c	Column depth, in. (mm)
d_p	Panel zone depth, measured from center of beam top flange to beam bottom flange, in. (mm)
d_z	Panel zone depth between continuity plates, in. (mm)
f_1	Load factor determined by the applicable building code for live loads but not less than 0.5
h_o	Height of braced element/beam, in. (mm)
h_{st}	Height of stiffener, in. (mm)
k	Distance for column detailing per AISC *Steel Construction Manual* design tables, in. (mm)
k	Stiffness of bracing element, kip/in. (N/mm)

$k1$	Distance for column detailing per AISC *Steel Construction Manual* design tables, in. (mm)
k_c	Distance from outer face of a column flange to web toe of fillet (design value) or fillet weld, in. (mm)
n	Number of braces
r_{yb}	Radius of gyration (beam), in. (mm)
s_h	Distance from the face of a column to a plastic hinge, in. (mm)
t	Distance from the neutral bending axis to the tension flange centroid, in. (mm)
t_{bf}	Thickness of beam flange, in. (mm)
t_{bw}	Thickness of beam web, in. (mm)
t_{cf}	Width of column flange, in. (mm)
t_{cf}	Minimum required thickness of column flange when no continuity plates are provided, in. (mm)
t_{cw}	Thickness of column web, in. (mm)
$t_{cont\text{-}pl}$	Thickness of the continuity plate in contact with the column flanges, in.^{2} (mm^2)
$t_{double\text{-}pl}$	Thickness of doubler plates, in. (mm)
t_{stiff}	Thickness of stiffener plate, in. (mm)
t_{zl}	Minimum panel zone thickness required by ANSI/AISC 341 (2005), in. (mm)
w	Uniform beam gravity load, kips per linear ft (N per linear mm)
w_{DL}	Factored uniform beam dead load, kips per linear ft (N per linear mm)
w_{LL}	Factored uniform beam live load, kips per linear ft (N per linear mm)
w_u	Factored uniform beam gravity load, kips per linear ft (N per linear mm)
w_z	Panel zone width between continuity plates, in. (mm)
β_b	Bending stiffness of brace element, kip/in. (N/mm)
β_{br}	Discreet brace stiffness, kip/in. (N/mm)
β_g	Stiffness of adjacent girders, kip/in. (N/mm)
β_{sec}	Secant stiffness of brace connection/stiffeners, kip/in. (N/mm)
β_T	Torsional brace stiffness, kip/in. (N/mm)
Δ_{rbe}	Displacement of relative brace element, in. (mm)
ϕ	Resistance factor
ϕ_d	Resistance factor for ductile limit states
ϕ_n	Resistance factor for nonductile limit states

1. Introduction



1.1 Introduction

Seismic building code provisions are based on two principal axioms: (1) provision of sufficient strength and stiffness in each element and the system as a whole to provide acceptable performance when subjected to ground shaking and (2) adequate detailing to achieve the required strength and stiffness. For connection design of steel special moment frame building systems complying with AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, 2005), the designer must use substantiating experimental test data (Appendix S of ANSI/AISC 341 [2005]) or a preapproved connection type, typically present in the standard AISC *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358, 2005). The connections described in the standard have been prequalified (per Appendix P of ANSI/AISC 341 [2005]) for use in special and intermediate steel moment frames without the need for additional testing, therefore explicitly complying with Appendix S of ANSI/AISC 341 (2005). ANSI/AISC 358 (2005) currently contains design and detailing requirements for three types of moment-resisting connections: reduced beam section (RBS) connections, unstiffened extended end plate (UEP) connections, and stiffened extended end plate (SEP) connections.

The design of the reduced beam section connection is discussed and presented herein.

1.2 Description of SMF

Structural design for large seismic events must explicitly consider the effects of response beyond the elastic range. The special moment frame (SMF) steel building system is designed such that the connections between the frame beams and columns absorb substantial energy through extensive rotational deformation, which is a major contributor to the displacement ductility capacity of the system.

Furthermore, an SMF lateral force resisting system is often preferred by building owners and architects due to the unobstructed program spaces afforded throughout the building.

After the discovery of brittle fractures in steel moment frame connections in the 1994 Northridge earthquake, structural engineers have three options for designing steel moment resisting frames:

- Conduct laboratory-based destructive tests acceptable to the building official (per Appendix S of ANSI/AISC 341 [2005]).
- Use prequalified column-to-beam connections per Appendix P (per ANSI/AISC 341 [2005]).
- Use patented connections that meet Appendix P and Appendix S requirements (per ANSI/AISC 341 [2005]).

The SAC Steel Project produced a common philosophy to design the SMF connection (particularly beam flange to column flange welds, which is an area of inherent variability and nonductility) such that the connection weld metal would remain elastic, thus forcing inelastic deformation of the beam connection to occur away from this highly restrained area.

1.3 Description of RBS

The fully restrained reduced beam section connection is recognized as a prequalified beam-column connection for use in a structural steel SMF. The removal of beam material at a short distance away from the column flange creates a “weak” area in the beam providing a reliable location for inelastic strain demand adequately distant from brittle weld metal. To create a predictable “weak” area, selective beam flange material is removed adjacent to the beam-column connection. The partial removal of beam flange reduces the cross-sectional area and moment capacity of the beam. Because of the moment gradient characteristic of SMF behavior, the beam plastic hinge forms within the reduced beam section (by design). The RBS connection obviates the addition of strengthening plates and special welding requirements to resist the expected moment capacity, M_{pe}, of the beam—typical of many post-Northridge moment connections.

The shape, size, and location of the RBS have an effect on the connection demand and performance. Various shapes have been tested and used in new construction. The prequalified RBS connection utilizes circular radius cuts in both the top and bottom flanges of the beam to reduce the flange area over a length of the beam near the ends of the beam span. Welds of beam flanges to column flanges are complete joint penetration groove welds per requirements of AWS D1.1 (2004), AWS D1.8 (2005), and ANSI/AISC 358 (2005). No reinforcement, other than weld metal, is used to join the flanges of the beam to the column. Web joints for SMF RBS connections are also constructed with complete penetration groove welds.

2. Summary of Research Findings, Limitations, and Requirements

2.1 Summary of Research Findings

Numerous experimental testing programs and analytical studies indicate that the RBS connection provides excellent energy dissipation and significant rotation capacity, two primary characteristics associated with “good” SMF seismic performance.

Most of the tested RBS connection assemblies have utilized beam spans of approximately 25 feet and beam depths ranging from W30 to W36. Beam span-to-depth ratios were typically in the range of eight to ten based on this data. Available RBS test data indicate that beam-connection assemblies, when designed and constructed according to the limits and procedures of AWS D1.1 (2004), AWS D1.8 (2005), and ANSI/AISC 358 (2005), have consistently developed rotational capacities of at least 0.04 radian under cyclic loading following pseudodynamic testing protocols. Tests show that yielding is generally concentrated within the reduced section of the beam. Peak strength of specimens is usually achieved at an interstory drift angle of 0.02 radian to 0.03 radian. Specimen strength gradually reduces due to local and lateral torsional buckling of the beam. Ultimate failure generally occurs at an interstory drift angle of 0.05 to 0.07 radian, typically because of low cycle fatigue fracture initiated by local flange buckling within the RBS.

2.2 Beam Limitations

The heaviest beam size for any tested RBS specimen is W36 × 300 as reported in FEMA-355D (2000). There is no evidence that modest deviations from the maximum tested specimen would result in considerably different performance. ANSI/AISC 358 (2005) limits maximum flange thickness for the connection to 1¾ inches, which is approximately 4% thicker than the flange of a W36 × 300.

ANSI/AISC 358 (2005), Section 5.3.1 outlines the following for the beam design:

- The beams shall be rolled wide-flange or built-up I-shape members.
- The maximum depth is that of a W36 section.
- The maximum weight is 300 lbs/ft.
- The clear span-to-depth ratio for SMF is 7 or greater.
- The width-to-thickness ratio for the beam flange can be calculated at a point located at the two-thirds point of the RBS cut.

- RBS connections that support a concrete structural slab and meeting requirements of ANSI/AISC 358 (2005) Section 5.3.1 are not required to have a supplemental brace at the RBS.
- If no floor slab is present, then a supplemental brace is required at the RBS. The brace should not be connected within the reduced section (protected zone; see Figure 3-1), but just outside (within d_b / 2) of the end of the radius cut, farthest from the face of the column.

2.3 Column Limitations

Almost all RBS connection test specimens have been fabricated with the beam flange welded to the column flange (that is, strong-axis connections). The limited amount of weak-axis testing has shown acceptable performance. However, ANSI/AISC 358 (2005) limits the prequalification to strong-axis connections only.

In FEMA-350 (2000, RBS connections were prequalified only for W12 and W14 columns. In ANSI/AISC 358 (2005), the prequalification of RBS connections is extended to include W36 columns. This extension of column size limit is primarily based on published research (Ricles et al. 2004). Research results indicate that adequate performance can be achieved with deep columns when a composite slab is present at the top of the frame beam or when adequate lateral bracing is provided for the beam and/or column in the absence of a slab.

ANSI/AISC 358 (2005) includes similar limits to column depths of cruciform columns. For built-up box columns, the deepest column tested in publicly available literature was 24 inches. Therefore, the limiting depth of built-up box columns for prequalified RBS connections is 24 inches. Limits on the width-thickness ratios for the walls of built-up box columns are specified in Section 2.3.2b(3) of AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, 2005). RBS connections are also prequalified for use with boxed wide-flange columns. When moment connections are made only to the flanges of the wide-flange portion of the boxed wide flange, the column may be up to W36 in depth.

2.4 Panel Zone Limitations

The minimum panel zone strength specified in Section 9.3a of the AISC *Seismic Provisions* (ANSI/AISC 341, 2005) is required for prequalified RBS connections. This requirement differs from recommendations presented in the FEMA-350 (2000).

2.5 Beam Flange to Column Flange Weld

Complete joint penetration groove welds joining the beam flanges to the column flanges are required to be made by the self-shielded flux cored arc welding process (FCAW-S) using electrodes with a minimum specified Charpy V-Notch (CVN) toughness. Three different electrode designations have commonly been used in successfully qualifying RBS connection tests: E71T-8, E70TG-K2, and E70T-6. Prequalified RBS connections do not require specific access-hole geometry, nor should special geometry be used to affect the SMF connection design. However, as a minimum, access holes must conform to the requirements of Figure C-J1.2 of the AISC specification (ANSI/AISC 360, 2005).

2.6 Beam Web to Column Connection

ANSI/AISC 358 (2005) requires a welded web connection (a complete joint penetration [CJP] weld) for RBS connections in structural steel SMF. In this welded connection, the beam web is welded directly to the column flange using a complete joint penetration groove weld. Bolted web connections are acceptable for use only in intermediate moment frames (IMFs).

2.7 Drift Requirement

Design story drift shall be calculated considering the softening effect of the RBS in the elastic frame response. The design story drift, calculated using the reduced frame stiffness, shall comply with limits specified in the applicable building code. For flange reductions up to 50% of the beam width, the effective elastic drifts calculated on gross beam cross section parameters may increase by 10% ($1.1 * \Delta_m$). Linear interpolation may be used for lesser values of beam width reduction. More rigorous analyses may determine a greater stiffness, resulting in reduced calculated elastic frame drifts.

3. RBS Design Procedure for SMF

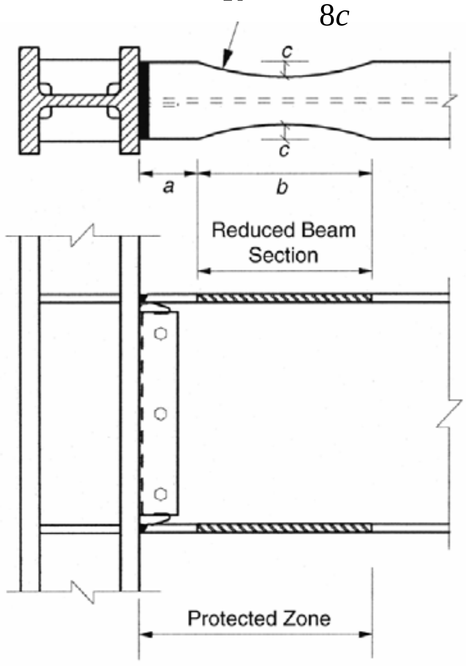
The design procedure and subsequent numerical example is limited to the examination of a representative reduced beam section (RBS) connection in a structural steel special moment frame (SMF) system. Gravity design and lateral design of the framing systems are not presented herein. Methods for frame design can be found in many textbooks, reference manuals, the SEAOC *Seismic Design Manual* (2006) and AISC *Seismic Design Manual* (2006). This design procedure may refer to intermediate moment frame (IMF) systems, but this design procedure is not intended for use in the design of an IMF system (but the results generated following this design procedure will also provide an adequate connection design for an IMF).

This RBS design procedure follows the requirements of ANSI/AISC 358 (2005), ANSI/AISC 341 (2005), and ANSI/AISC 360 (2005).

Step 1: Choose plastic hinge configuration and location

The intent of the reduced beam section is to move the plastic hinge region away from the weld between the beam flange and column flange (Figure 3-1). This is accomplished by reducing the beam's actual plastic moment by removing part of the beam flange. This reduced section creates a weaker location where yielding and plastic hinge formation are expected to occur.

$$R = \frac{4c^2 + b^2}{8c}$$



(Figure from ANSI/AISC 358 [2005])

Figure 3-1: RBS Geometry

a) ANSI/AISC 358 (2005), Section 5.8, prescribes limitations for the dimensions of the radius cut for the reduced beam section as follows:

$0.5b_{bf} \leq a \leq 0.75b_{bf}$ (ANSI/AISC 358 [2005], 5.8-1)

$0.65d_b \leq b \leq 0.85d_b$ (ANSI/AISC 358 [2005], 5.8-2)

$0.1b_{bf} \leq c \leq 0.25b_{bf}$ (ANSI/AISC 358 [2005], 5.8-3)

b) The radius of the flange cut can be calculated as follows:

$$R = \frac{4c^2 + b^2}{8c}$$

c) The plastic hinge may be assumed to occur at the center of the curved cut a distance S_h away:

$$S_h = d_c / 2 + a + b / 2$$

d) The distance between plastic hinges are:

$$L' = L_o - 2(S_h)$$

The length between the plastic hinges L' (Figure 3-2) is used to determine the demands at the critical sections for the connection analysis.

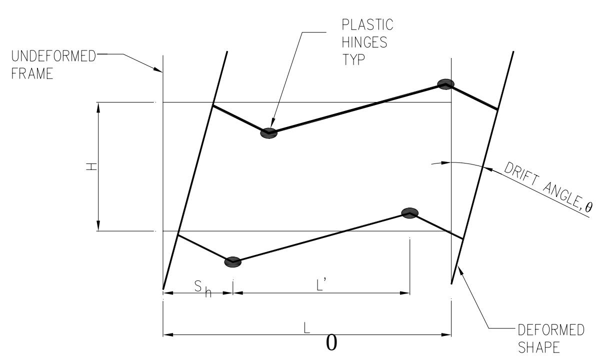


Figure 3-2: Plastic Hinge Locations

Step 2: Determine plastic section modulus at the reduced beam section

The plastic section modulus at the center of the reduced beam section is a function of the initial plastic section modulus of the beam minus the portion removed and is calculated as follows:

$Z_e = Z_{xb} - 2ct_{bf}\left(d_b - t_{bf}\right) = Z_{RBS}$ (ANSI/AISC 358 [2005], Equation 5.8-4)

Step 3: Determine probable maximum moment at the reduced beam section

The probable plastic moment at the center of the reduced beam section M_{pr} is calculated as:

$$M_{pr} = C_{pr} R_y F_y Z_e \qquad \text{(ANSI/AISC 358 [2005], 2.4.3-1, 5.8-5)}$$

The factor C_{pr} is an estimation of the maximum connection strength expected, including strain hardening, local restraint, additional reinforcement, and other connection conditions.

$$C_{pr} = \frac{F_y + F_u}{2F_y} \leq 1.2 \qquad \text{(ANSI/AISC 358 [2005], 2.4.3-2)}$$

The value for M_{pr} must be such that the projected moment demand at the face of the column, M_f, is less than the expected strength of the full beam section; this condition is verified in Step 7.

Step 4: Compute the shear force at the center of each RBS

The shear force is calculated at the center of the reduced beam sections located near each end of the beam. ANSI/AISC 358 (2005) Section 5.8 requires that this shear force be determined by a free body diagram of the portion of the beam between the centers of the reduced beam sections, which assumes that the moment at the center of each RBS is M_{pr}. The free body diagram also includes the gravity loads acting on the beam based on the following load combination: 1.2*DL* + f_1*LL* + 0.2*SL* (where f_1 = 0.5). Figure 3-3 represents a theoretical free body diagram for beam shear.

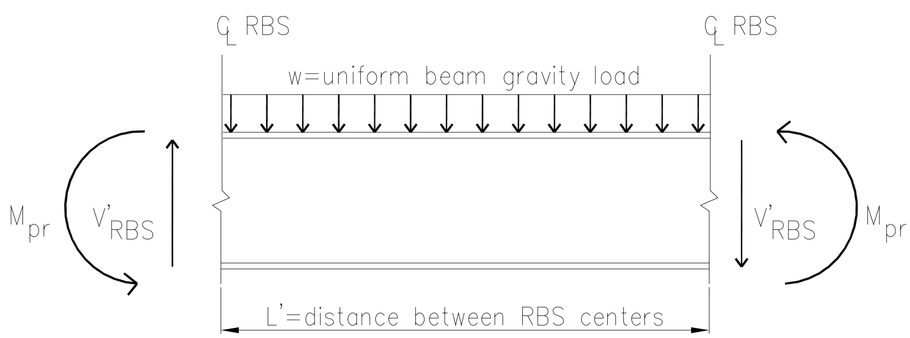


Figure 3-3: Beam Equilibrium under the Probable Plastic Moment M_{pr}

The equation used to calculate the shear force at the center of the reduced beam section follows:

$$V_{RBS} = 1.2(V_{DL}) + 0.5(V_{LL}) + (V_{pr})$$

Where the shear due to the plastic moment capacity of the RBS is given by the following:

$$V_{pr} = \frac{2M_{pr}}{L'}$$

Note that this shear can be positive or negative in respect to the gravity load. Therefore, the shear forces at the center of the RBS can be calculated as follows:

$$\therefore V_{RBS} = 1.2(V_{DL}) + 0.5(V_{LL}) + (V_{pr}) \qquad = V_{RBS} \quad \text{(positive sense)}$$

$$\text{And } V_{RBS} = 1.2(V_{DL}) + 0.5(V_{LL}) - (V_{pr}) \qquad = V'_{RBS} \quad \text{(negative sense)}$$

Step 4 (Figure 3-3) considers a beam with a uniformly distributed gravity load. For gravity load conditions other than a uniform load, appropriate adjustments should be made to the free body diagram and to the equations above. In addition, the equations included in Step 4 assume that plastic hinges will form within the RBS at each end of the beam. If the gravity load on the beam is very large, the plastic hinge may move relative to the center of the RBS.

Step 5: Compute the probable maximum moment at the face of the column

The moment at the face of the column is computed as:

$$M_f = M_{pr} + V_{RBS} s_h \qquad \text{(ANSI/AISC 358 [2005], Equation 5.8-6)}$$

Where $s_h = a + b/2$

In this example (as in many typical applications) the moment due to the gravity load applied between the plastic hinge and the face of the column flange is negligible (much less than 0.5% of M_{pr}); therefore, the moment due to gravity may be omitted from the connection design with negligible consequences.

Check $M_{gravity} = \dfrac{w_u (s_h)^2}{2}$ kip-in. << 0.5% of M_f

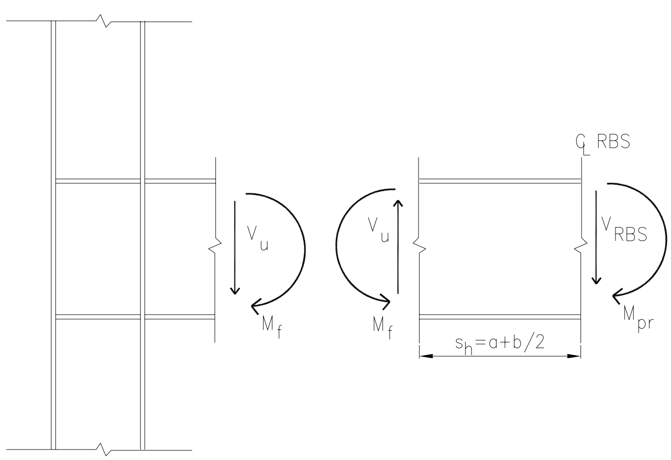


Figure 3-4: Free Body Diagram between Center of RBS and Face of Column

Step 6: Compute the expected plastic moment of the beam

M_{pe}, the expected plastic moment of the beam, is based on the expected yield stress of the beam material and is computed as follows:

$$M_{pe} = Z_{xb} R_{yb} F_{yb} \qquad \text{(ANSI/AISC 358 [2005], Equation 5.8-7)}$$

Step 7: Check that M_f does not exceed $\phi_d M_{pe}$

Check the moment at the face of the column, M_f, against the plastic moment of the beam, $\phi_d M_{pe}$, from Step 6, as follows:

$$M_f / \phi_d M_{pe} \leq 1.0 \qquad \text{(ANSI/AISC 358 [2005], Equaton 5.8-8)}$$

Per ANSI/AISC 358 (2005), $M_f / \phi_d M_{pe}$ should be in the range of 85% to 100% of M_{pe}.

If M_f exceeds $\phi_d M_{pe}$:

a) Increase the depth of cut at the reduced beam section "c", not to exceed 25% of b_f (Figure 3-1);

b) and/or decrease the values of a and b;

c) and/or select a different beam.

For ductile limit states, as in the reduced beam section connection, the resistance factor per ANSI/AISC 358 (2005) may be taken as 1.0:

$$\phi_d = 1.0$$

Step 8: Determine the required shear strength

Determine the required shear strength of the beam and beam web to column connection from the following equation:

$$V_u = \frac{2M_{pr}}{L'} + V_{gravity} \qquad \text{(ANSI/AISC 358 [2005], Equation 5.8-9)}$$

$$= V_{pr} + V_{gravity} = V_{RBS} \qquad \text{(Shear at the RBS)}$$

Prior to checking the design bending and shear strengths of the member, SMF beams must be checked for stability and proportions per ANSI/AISC 358 (2005), Section 5.3, with references to ANSI/AISC 341 (2005), Section 9.8, which limits the width-thickness ratios for elements subject to compression forces. Calculations for these checks are not included in this procedure or in the following numerical example, as the equations are available in AISC *Commentary* (ANSI/AISC 341, 2005).

The design shear strength of the beam is checked in accordance with ANSI/AISC 360 (2005), Chapter G:

$$d_b / t_{bw} < 2.45\sqrt{E/F_y}$$ (ANSI/AISC 341 [2005], Table I-8-1)

$$\therefore V_n = 0.6F_y A_w C_v$$ (ANSI/AISC 360 [2005], G2-1)

Where $C_v = 1.0$ (ANSI/AISC 360 [2005], G2-2)

Step 9: Design the beam web to column connection

ANSI/AISC 358 (2005), Section 5.6, indicates that the strength of the beam web to column connection strength must be greater than the ultimate design shear value determined with Equation 5.8-9 (see Step 8). In addition, the following description identifies the only allowable detailing for the beam web to column connection in SMF systems:

> For SMF systems, the beam web shall be connected to the column flange using a CJP groove weld extending between weld access holes. The single plate shear connection shall be permitted to be used as backing for the CJP groove weld. The thickness of the plate shall be at least 3/8 in. (10 mm). Weld tabs are not required at the ends of the CJP groove weld at the beam web. Bolt holes in the beam web for the purpose of erection are permitted.

Because the beam web to column connection is made with a complete joint penetration (CJP) groove weld, the shear capacity of the weld is greater than or equal to the shear capacity of the beam assuming standard weld access holes per ANSI/AISC 360 (2005), Section J. For SMF systems, no further checks are required to verify the adequacy of this connection.

Note that for IMF systems, ANSI/AISC 358 (2005) permits the use of a bolted single plate shear connection to connect the beam web to the column flange. In the case of the IMF, the connection will have to be checked for the shear demand calculated in Step 8.

Step 10: Check continuity plate requirements

Per ANSI/AISC 358 (2005), Chapter 2, Section 2.4.4:

> When the beam flange connects to the flange of a wide-flange or built-up I-shaped column having a thickness that satisfies Equations 2.4.4-1 and 2.4.4-2, continuity plates need not be provided:

$$t_{cf} \geq 0.4\sqrt{1.8 b_{bf} t_{bf} \frac{F_{yb} R_{yb}}{F_{yc} R_{yc}}} \qquad \text{(ANSI/AISC358[2005], 2.4.4-1)}$$

$$t_{cf} \geq \frac{b_{bf}}{6} \qquad \text{(ANSI/AISC358[2005], 2.4.4-2)}$$

Where continuity plates are required, ANSI/AISC 358 (2005), Section 2.4.4a, says that the thickness of the plates is determined as follows:

> a) For one-sided (exterior) connections, continuity plate thickness shall be at least one-half of the thickness of the beam flange.
>
> b) For two-sided (interior) connections, the continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column.

Continuity plates shall also conform to the requirements of Section J of ANSI/AISC 360 (2005), which prescribes certain requirements for detailing and sizing of the continuity plates. ANSI/AISC 358 (2005), Section 3.6, indicates the following detailing provisions for the welding of the continuity plates:

> Along the web, the corner clip shall be detailed so that the clip extends a distance of at least 1-1/2 in. (38 mm) beyond the published "k" detail dimension for the rolled shape. Along the flange, the plate shall be clipped to avoid interference with the radius of the rolled shape and shall be detailed so that the clip does not exceed a distance of 1/2 in. (12 mm) beyond the published "k1" detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. When a curved clip is used, it shall have a minimum radius of 1/2 in. (12 mm).

Using these requirements, in conjunction with the requirements of AWS D1.8 (2005), the projected contact area between the edge of the continuity plate and the column flange is calculated as follows:

$$A_{pb} = W_{pb-flange} t_{cont-pl}$$

$$W_{pb-flange} = b_{cont-pl} - (\text{"}k1_{col}\text{"} + 0.25 in.)$$

The continuity plate width ($W_{pb-flange}$) can be determined using the following equation:

$$W_{pb-flange} = b_{cont-pl} - (\text{"}k1_{col}\text{"} + 0.25in.)$$

The maximum contact area between the continuity plate and the column web is:

$$\text{For: } A_{pb} = W_{pb-flange} t_{cont-pl}$$

The continuity plate thickness can be determined using requirements of ANSI/AISC 358 (2005) Section 2.4.4c (thickness of the thinnest beam flange framing into a column flange). In addition to the size of the continuity plate, the attachment/welding of the continuity plate shall meet the criteria established in ANSI/AISC 358 (2005), Section 2.4.4b, and AWS D1.8 (2005) (for weld quality).

Per ANSI/AISC 358 (2005), Section 2.4.4b, continuity plates shall be welded to column flanges with CJP groove welds, so no design calculations are required for this portion of the connection. However, the connection between the continuity plate and column web should be calculated to determine an appropriate weld for this connection.

Continuity plates shall be welded to column webs using groove welds or fillet welds. The required strength of the sum of the welded joints of the continuity plates to the column web shall be the smallest of the following:

a) The sum of the design strengths in tension of the contact areas of the continuity plates to the column flanges that have attached beam flanges. (The tension strength of the continuity plate is limited by the connection strength between the edge of the continuity plate and the inside face of the column flange.)

b) The design strength in shear of the contact area of the plate with the column web.

c) The design strength in shear of the column panel zone.

d) The sum of the expected yield strengths of the beam flanges transmitting force to the continuity plates.

The maximum contact area between the continuity plate and the column web is:

$$A_{pw} = t_{cont-pl}[d_c - 2t_{cf} - 2(k + 1.5in.)] \qquad \text{per continuity plate}$$

The following calculations identify the controlling design strength for the continuity plate to column web welds:

a) $\sum \phi F_y A_{pb}$ (ANSI/AISC 358 [2005], 2.4.4b (a))

b) $\phi_v V_{nw} = \phi(0.6)F_y A_{pw}$ (ANSI/AISC 358 [2005], 2.4.4b (b))

c) ϕR_v Panel Zone Shear strength (See Step 11 and ANSI/AISC 358 [2005], 2.4.4b (c))

d) $\sum \frac{\phi_d M_{pe}}{d_b - t_{bf}}$ (ANSI/AISC 358 [2005], 2.4.4b (d))

The smallest of a) through d) above is used to design the welds between the continuity plate and the column web.

The minimum required double-sided fillet weld size (if used) per continuity plate is calculated as follows:

$$D_{\min} = \frac{R_{ct-cw}}{2(1.392k/in.)[d_c - 2t_{cf} - 2(k + 1.5in.)]}$$

D_{min} can also be used to size a partial joint penetration (PJP) groove weld when continuity plate thickness is adequately larger than D_{min}.

Step 11: Check column panel zone

The panel zone strength is calculated per ANSI/AISC 341 (2005), Section 9.3. The free body diagram is shown in Figure 3-5.

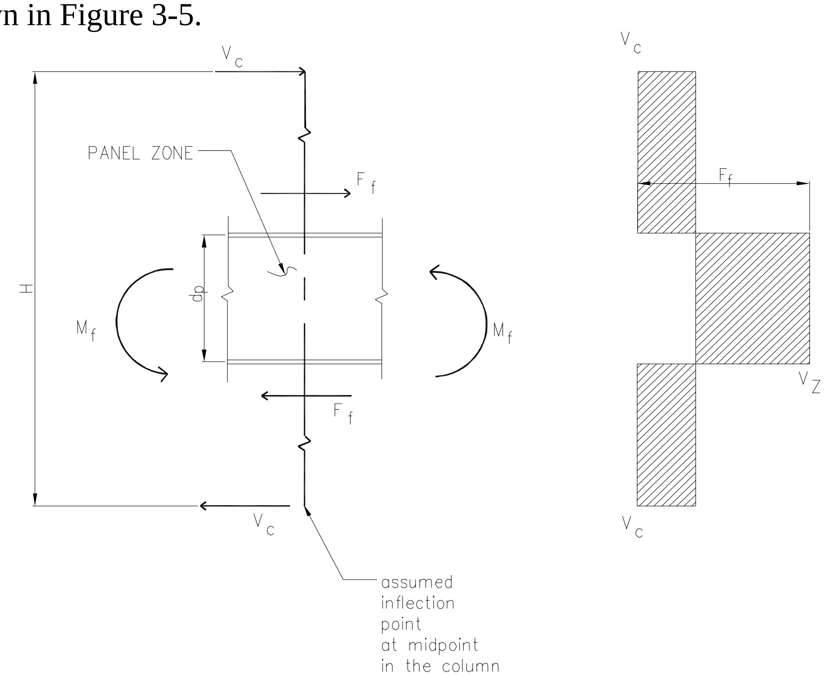


Figure 3-5: Panel Zone Forces

$$d_p = d_c - t_{cf}$$

$$M_f = M_{pr} \pm V_{RBS} s_h \qquad \text{(From Step 5)}$$

$$R_u = F_f = \frac{\sum M_f}{d_p}$$

The panel zone shear strength is determined from ANSI/AISC 341 (2005), Section 9.3a, which refers to ANSI/AISC 360 (2005), Section J10.6:

$$\phi R_v = \phi 0.6 F_{yc} d_c t_w \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right] \qquad \text{(ANSI/AISC 360 [2005], J10-11)}$$

where:

$\phi = 1.0$ per ANSI/AISC 341 (2005), Section 9.3a

$t_w = t_{cw} + t_{double\text{-}pl}$

If R_u is $< \phi R_v$, doubler plates are required.

The minimum panel zone thickness t_z is also checked per ANSI/AISC 341 (2005), Section 9.3b:

$$t_z \geq (d_z + w_z)/90 \qquad \text{(ANSI/AISC 341 [2005], 9-2)}$$

where:

d_z = panel zone depth between continuity plates
w_z = panel zone width between column flanges

When doubler plates are used in lieu of increasing the column size, compliance with ANSI/AISC 341 (2005), Section 9.3c is required.

Step 12: Check column beam moment ratio

ANSI/AISC 358 (2005), Section 5.4, references ANSI/AISC 341 (2005), Sections 9.3 and 10.3 for SMF and IMP systems, respectively, and requires that all SMF connections satisfy the following:

$$\sum M_{pc}^* / \sum M_{pb}^* > 1.0 \qquad \text{(ANSI/AISC 341 [2005], 9-3)}$$

The axial load on the column must be taken into account when determining the flexural strength of the column at the beam centerline.

$$\sum M_{pc}^* = \sum [Z_{xc}(F_{yc} - P_{uc} / A_g)]$$

$$\sum M_{bc}^* = \sum (1.1 R_y F_{yb} Z_{RBS} + M_v)$$

where $\sum M_v = (V_{RBS} + V'_{RBS})(S_h)$

The beam-column strength ratios (or strong column weak beam) must be satisfied.

Step 13: Check lateral bracing of columns

Per ANSI/AISC 358 (2005), Section 5.3.2, lateral bracing of columns shall conform to Section 9.7 (or 10.7) for SMF (or IMF) in the AISC seismic provisions (ANSI/AISC 341, 2005) to prevent the SMF column members from experiencing lateral torsional buckling. Section 9.7a of ANSI/AISC 341 (2005) allows the use of a strong column weak beam ratio $\Sigma M_{pc}^*/\Sigma M_{pb}^*$ (SCWB—determined in Step 12) greater than 2.0 to show that a column remains elastic outside of the panel zone at restrained beam to column connections. If the SCWB is greater than 2.0, Section 9.7a requires that the column flanges be braced at the level of the beam top flanges only (typically provided by a floor slab system).

In cases where the SCWB is less than 2.0, column flanges must be braced at both beam flanges. At the beam top flange, the concrete slab effectively provides bracing for the column flange. The column flanges therefore need to be laterally braced at the moment beam bottom flange only.

ANSI/AISC 341 (2005), Section 9.7a, indicates that the column flange brace, when provided, must have strength equal to 2% of the available beam flange strength.

$$P_{br} = 0.02 F_{yb} b_{bf} t_{bf}$$

Step 14: Check beam lateral bracing requirements

At M_{pr}, the bottom flange of the frame beam will tend to move laterally (lateral torsional buckling behavior). ANSI/AISC 360 (2005), Appendix 6.3, indicates that lateral stability of frame beams may be provided by lateral bracing, or torsional bracing, or a combination of the two.

Attachment of lateral bracing to the frame beam at the plastic hinges is located at a distance no greater than d_b / 2 beyond the end of the reduced beam section farthest from the face of the column (per ANSI/AISC 358 (2005), Section 5.3.1). ANSI/AISC 358 (2005), Section 5.3.1.7, provides an exception for beams supporting a concrete structural slab that is connected between the protected zones with welded shear connectors spaced at a maximum of 12 inches on center. For beams supporting a concrete structural slab, the beam is not required to have supplemental top and bottom flange bracing at the RBS. The protected zone is defined to be the region of the beam between the face of the column and the end of the reduced beam section cut farthest from the face of the column. In these cases, brace spacing is determined per ANSI/AISC 341 (2005), Section 9.8.

Exploration of beam bracing is presented in three cases: Case A uses a lateral brace to provide lateral stability to the frame beam, Case B uses a torsional brace, and Case C considers bracing a flying beam condition.

For lateral stability of the frame beam, ANSI/AISC 358 (2005), Section 5.3.1(7), which references ANSI/AISC 341 (2005) Section 9.8, gives a maximum brace spacing, L_{br}, which is defined as follows:

$$L_{br} = \frac{0.086 r_{yb} E}{F_{yb}}$$

<u>Case A: Nodal bracing provided by angle to adjacent parallel gravity beam—lateral brace</u>

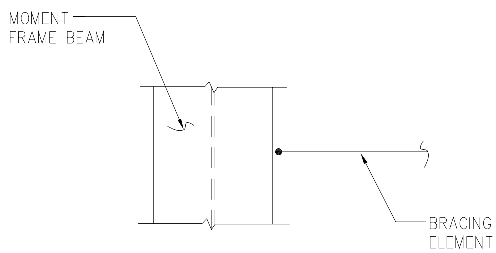


Figure 3-6: Case A—Beam Lateral Bracing

Case A describes a lateral bracing element, such as an angle connected between the bottom flange of the moment frame beam to the top flange of an adjacent gravity beam.

ANSI/AISC 341 (2005), C9.8, indicates that if lateral braces are provided adjacent to the plastic hinges, they should provide a design strength of at least 6% of the expected internal force associated with the plastic moment capacity of the beam flange at the plastic hinge location. This value can be used to design bracing elements located along the length of the beam. The minimum required force is 2% of the expected internal force associated with the plastic moment capacity of the beam flange. If an RBS connection detail is used, the reduced flange width may be considered in calculation of the bracing force.

$$P_{br} = 0.02 \frac{M_{pr} C_d}{h_o} \qquad \text{(ANSI/AISC 360 [2005], A-6-7)}$$

$$P_{br} = 0.06 \frac{M_{pr} C_d}{h_o} \text{ (for conditions where beam not supporting slab)}$$

(ANSI/AISC 341 [2005] Section 9.8)

The length of the brace is assumed to be measured from the centerline of the moment frame beam to the centerline of the adjacent gravity beam or end of bracing element. The bracing element, with consideration to unbraced length, shall be designed to resist this bracing force.

ANSI/AISC 341 (2005), Appendix 6, also requires a minimum stiffness for the bracing element. The required brace stiffness is calculated as a direct horizontal stiffness using the following equation:

$$\beta_{br} = \frac{10 M_r C_d}{\phi L_b h_o} \qquad \text{(ANSI/AISC 360 [2005], A-6-8)}$$

where:

$\phi = 0.75$
$M_r = M_{pr}$
C_d = 1.0 for single curvature; 2.0 for reverse curvature; C_d = 2.0 only applies to the brace closest to the inflection point
$h_o = d_b - t_{bf}$
L_b = unbraced length of the moment beam (between points of lateral bracing)

The stiffness of the bracing element can be calculated as:

$$k = \frac{A_g E}{L_{brace}} \cos^2(\theta)$$

k must be $> \beta_{br}$

Case B: Nodal bracing provided by perpendicular beam—torsional brace

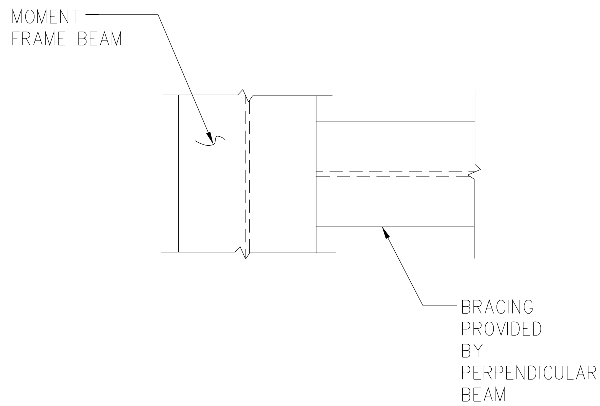


Figure 3-7: Case B—Beam Torsional Bracing

Case B presents the perpendicular bracing beam as a torsional brace. Therefore the perpendicular beam must be designed with adequate torsional strength and stiffness. Note that the perpendicular beam may also be designed as a lateral brace (presented in Case A). The lateral and torsional demands are transmitted to the bracing beam through a full depth stiffener, to prevent warping and fully brace the moment beam cross section. The bracing beam is then designed to resist the applied moment.

Derivations for the torsional strength and stiffness requirements can be found in papers by Yura and Helwig (1999) and Yura (2001) and are summarized in ANSI/AISC 360 (2005), Appendix 6. The following are the governing equations:

$$M_{br} = \frac{0.024 M_r L_{pbm}}{n C_{bb} L_b} \qquad \text{(ANSI/AISC 360 [2005], A-6-9)}$$

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{\sec}}\right)} \qquad \text{(ANSI/AISC 360 [2005], A-6-10)}$$

$$\beta_T = \frac{2.4 L_{pbm} M_f^{\ 2}}{\phi (n E I_{eff} C_b^{\ 2})} \qquad \text{(ANSI/AISC 360 [2005], A-6-11)}$$

$$\beta_{\sec} = 3.3 \frac{E}{d_b - t_{bf}} \left[\frac{1.5 (d_b - t_{bf}) t_{bw}^{\ 3}}{12} + \frac{t_{stiff} b_{stiff}^{\ 3}}{12} \right] \qquad \text{(ANSI/AISC 360 [2005], A-6-12)}$$

Where:

$\phi = 0.75$
L_{pbm} = length of the perpendicular bracing beam
L_b = length of moment beam between points of torsional bracing
$M_r = M_{pr}$ (of SMF beam)
I_y = out of plane moment of inertia
n = number of nodal brace points in span
C_b = moment diagram modification factor for the full bracing condition
(or effectively braced beam per ANSI/AISC 360 [2005], Chapter F)

<u>Case C: Relative bracing (flying beam condition)—brace along length of frame beam</u>

Case C is provided as an illustration for conditions requiring lateral bracing of the frame beam along the length of the beam (between the reduced beam sections) by the use of a brace element parallel to the length of the beam. Case C illustrates a condition where a perpendicular beam or a brace cannot be attached to an adjacent gravity frame beam. An example would be a frame beam at an elevator shaft at the exterior edge of the structure. In this condition, the brace parallel to the beam is defined as a relative brace. In Case C, a combination of lateral bracing and torsional bracing is used to provide lateral stability of the SMF frame beam.

The relative brace may be considered as a nodal brace with forces transferred from the SMF frame beam into the relative brace through discrete connection points. Since the connection between the relative brace and the moment beam must be strong and stiff enough to engage the relative brace, the connection design is based on demands for a nodal brace (for the SMF beam) using equations for torsional bracing of the SMF beam per methods shown in Case B. The relative brace is then designed considering the relative brace equations in ANSI/AISC 360 (2005), Appendix 6.

In the case of the beam lateral/torsional connection, restraint against rotation about the beam in the longitudinal axis is considered so AISC equation A-6-7 applies.

$$P_{br} = 0.02\frac{M_r C_d}{h_o} \qquad \text{(ANSI/AISC 360 [2005], A-6-7)}$$

Where C_d = 1.0 for single curvature, 2.0 for double curvature

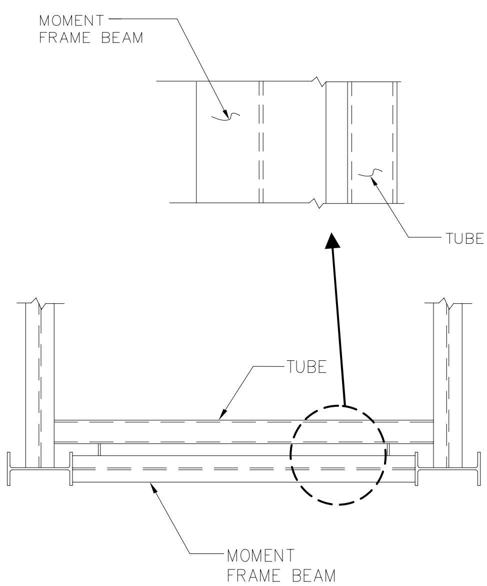


Figure 3-8: Case C—Beam Relative Bracing

Back-calculating β_{br}, from ANSI/AISC 360 (2005), A-6-8, with the common factor of $\frac{M_r C_d}{h_o}$, $\beta_{br} = \frac{10}{\phi L_b}(50P_{br})$. L_b, in this case, is the unbraced length of the relative brace. Set $\beta_{br} = P_{br} / \Delta_{rbr}$

to estimate a required stiffness for the connection between the frame beam and the relative brace element.

To check the adequacy of the relative brace:

$$P_{br} = 0.008\frac{M_r C_d}{h_o} \qquad \text{(ANSI/AISC 360 [2005], A-6-5)}$$

$$\beta_{br} = \frac{4 M_r C_d}{\phi L_b h_o} \qquad \text{(ANSI/AISC 360 [2005], A-6-6)}$$

This calculation is further illustrated in the numerical design example.

4. RBS Numerical Design Example

Background information for the following numerical design example is presented in Sections 4.1 and 4.2. This information is provided as content for the RBS connection design only.

4.1 Description of Design Example Project

a) Governing Building Code: 2006 International Building Code (ICC 2006)
b) Telecommunications Building (Essential)—Importance Factor, $I = 1.5$
c) Lateral Force Resisting System—Special Moment Resisting Frame, $R = 8$
d) Soil Profile Type = S_d (default value)
e) Maximum considered earthquake (MCE) spectral response acceleration
    (5% damped, site class B)
    $S_S = 1.5g$ @ $T = 0.2$ sec Figure 1615 (3)
    $S_1 = 0.6g$ @ $T = 1.0$ sec Figure 1615 (4)
    Sources of MCE spectral response accelerations:
    IBC (2006) — Maps presented in Figure 1613.5(1) through Figure 1613.5(14)
    ASCE 7 (2005) — Figure 22-1 through Figure 22-14
    USGS website - www.usgs.gov

4.2 Description of Design Example Building

a) Building description:

8-bay by 8-bay, rectangular in plan
Centerline spans of 28 feet
5-story structure above grade (no basement, no mezzanines, no penthouse)
Typical floor heights of 15 feet and 16 feet at the first floor
4-bay moment frames on all four perimeter sides of the building
No interior moment frames

b) Modeling assumptions:

Pinned base
Centerline model with panel zone set to 0%
Maximum allowable ~ 90% of code allowable (see drift discussion in Section 2.7)

c) Loading:

Dead and live loads modeled as uniform area loads
Activated self-weight for members
Curtain walls modeled as perimeter line loads

d) Factored gravity loads on beam:

$w_u = 1.2(w_{DL}) + 0.5(w_{LL}) = 2$ k/ft
$V_{gravity} = 1.2(V_{DL}) + 0.5(V_{LL}) = 28$ kips

e) Beam and column sizes:

With consideration to strength and drift limitations, strong column/weak beam and panel zone strength criteria, the following beam and column sizes at the first-story level were selected at one of the perimeter moment frames for this design example. The selected sizes illustrate the various calculations outlined in the design procedure. A more cost-effective design might utilize a larger column to avoid the need for continuity plates and doubler plates.

Beam:	W36 × 282
	A992
	F_{yb} = 50 ksi
Column:	W36 × 395
	A992
	F_{yc} = 50 ksi

f) Section properties:

W36 × 282

d_b	=	37.1	in.
b_{bf}	=	16.6	in.
t_{bf}	=	1.57	in.
t_{bw}	=	0.885	in.
Z_{xb}	=	1,190.0	in.^{3}
r_{by}	=	3.8	in.
I_{by}	=	1,200.0	in.^{4}

W36 × 395

d_c	=	38.4	in.
b_{cf}	=	16.8	in.
t_{cf}	=	2.2	in.
t_{cw}	=	1.22	in.
Z_{xc}	=	1,710.0	in.^{3}
A_g	=	116.0	in.^{2}

4.3 Connection Design

As noted previously, this numerical example considers only the RBS connection design. Gravity design and lateral design of the steel moment frame itself is not presented herein.

Step 1: Choose plastic hinge configuration and location

a) Per ANSI/AISC 358 (2005), Section 5.8:

$$0.5b_{bf} = 0.5(16.6) = 8.3 \text{ in.}$$

$$0.75b_{bf} = 0.75(16.6) = 12.5 \text{ in.}$$

$\therefore$ Use $a = 10.0$ in. and,

$$0.65d_b = 0.65(37.1) = 24.1 \text{ in.}$$

$$0.85d_b = 0.85(37.1) = 31.5 \text{ in.}$$

$\therefore$ Use $b = 28.0$ in.

With a 45% reduction in the flange area:

$$c = 0.45\left(\frac{b_{bf}}{2}\right) = \frac{0.45(16.6)}{2} = 3.74 \text{ in.}$$

$\therefore$ Use <u>c = 3.75 in.</u> cut, [3.75 in. / 16.6 in.] = $0.226b_f < 0.25b_f$ *OK*

b) Determine the radius of the flange cut:

$$R = \frac{4c^2 + b^2}{8c} = \frac{4(3.75)^2 + 28.0^2}{8(3.75)} = 28 \text{ in.}$$

c) Determine distance to RBS:

$$S_h = d_c/2 + a + b/2 = (38.4/2) + 10.0 + (28/2) = 43.2 \text{ in.}$$

d) Determine distance between plastic hinges:

$$L_o = 28.0 \text{ ft}$$

$$\therefore L' = 28 - 2(43.2/12) = 20.8 \text{ ft}$$

<u>Step 2: Determine plastic section modulus at the reduced beam section</u>

$$Z_e = Z_{xb} - 2ct_{bf}\left(d_b - t_{bf}\right) = Z_{RBS}$$ (ANSI/AISC 358 [2005], Equation 5.8-4)

$$= 1{,}190 - [2(3.75)(1.57)(37.1 - 1.57)] = 771.6 \text{ in}^3$$

<u>Step 3: Determine probable maximum moment at the reduced beam section</u>

$$C_{pr} = \frac{Fy + Fu}{2Fy} = \frac{50ksi + 65ksi}{2(50ksi)} = 1.15 \leq 1.2$$ (ANSI/AISC 358 [2005], 2.4.3-2)

$$M_{pr} = C_{pr}R_yF_{yb}Z_e = (1.15)(1.1)(50)(772) = 48{,}829 \text{ kip-in.}$$

(ANSI/AISC 358 [2005], 2.4.3-1, 5.8-5)

Step 4: Compute the shear force at the center of each RBS

Shear due to the M_{pr} :

$$V_{pr} = \frac{2M_{pr}}{L'} = \frac{2(48{,}829)}{12(20.8)} = 391 \text{ kips}$$

Shear at the reduced beam sections:

$$V_{RBS} = V_{gravity} \pm (V_{pr})$$

$$V_{gravity} = 1.2(V_{DL}) + 0.5(V_{LL}) = 28 \text{ kips}$$

$$\therefore V_{RBS} = 28 + (391) = 419 \text{ kips} \qquad \text{(positive sense)} \qquad \textit{\textbf{controls the design}}$$

$$\text{And } V_{RBS} = 28 + (-391) = -363 \text{ kips} \qquad \text{(negative sense)}$$

Step 5: Compute the probable maximum moment at the face of the column

The moment at the face of the column is:

$$M_f = M_{pr} + V_{RBS} s_h \qquad \text{(ANSI/AISC 358 [2005], Equation 5.8-6)}$$

$$M_f = 48{,}829 + 419\left(10 + \tfrac{28}{2}\right) = 58{,}885 \text{ kip-in.} \qquad \text{(positive sense)} \qquad \textit{\textbf{controls the design}}$$

$$M'_f = -48{,}806 + -363\left(10 + \tfrac{28}{2}\right) = -57{,}518 \text{ kip-in.} \qquad \text{(negative sense)}$$

Check:

$$M_{gravity} = (1/2) w_u (s_h)^2 = (1/2)\frac{(2kip/ft)}{12in/ft}(10 + 28/2)^2 = 48.0 \text{ kip-in.} <<0.5\% \text{ of } M_f$$

Step 6: Compute the expected plastic moment of the beam

$$M_{pe} = Z_{xb} R_y F_{yb} = (1{,}190)(1.1)(50) = 65{,}450 \text{ kip-in.}$$

(ANSI/AISC 358 [2005], Equation 5.8-7)

Step 7: Check that M_f does not exceed $\phi_d M_{pe}$

Check the value of M_f against $\phi_d M_{pe}$ as follows:

$$M_f / \phi_d M_{pe} = 58{,}885/1.0(65{,}450) = 0.9 < 1.0 \quad OK$$

(ANSI/AISC 358 [2005], Equation 5.8-8)

The results of Steps 2 through 7 indicate that no change is required for the dimensions of the reduced beam section assumed in Step 1. The following calculations determine the adequacy of the frame beam and the frame column within the connection.

Step 8: Determine the required shear strength

The demand at the RBS was determined in Step 4:

$$V_u = \frac{2M_{pr}}{L'} + V_{gravity} = V_{pr} + V_{gravity} = V_{RBS} = 419 \text{ kips}$$

(ANSI/AISC 358 [2005], Equation 5.8-9)

The design shear strength of the beam is checked in accordance with ANSI/AISC 360 (2005), Chapter G:

$$d_b / t_{bw} = 41.9 < 2.45\sqrt{29{,}000/50} = 59.0$$ (ANSI/AISC 341 [2005], Table I-8-1)

$$\therefore V_n = 0.6F_{yb}A_wC_v = 985 \text{ kips}$$ (ANSI/AISC 360 [2005], G2-1)

Where $C_v = 1.0$ (ANSI/AISC 360 [2005], G2-3)

Therefore, the beam is adequate to resist the shear demand at any location along the beam length.

Step 9: Design the beam web to column connection

The beam web to column connection is made with a complete joint penetration (CJP) groove weld. Therefore, shear capacity of the weld is greater than or equal to the shear capacity of the beam. No additional checks are required to verify the adequacy of this connection based on the results of Step 8.

Step 10: Check continuity plate requirements

Check whether continuity plates are required:

$$t_{cf} \geq 0.4\sqrt{1.8b_{bf}t_{bf}\frac{F_{yb}R_{yb}}{F_{yc}R_{yc}}} = 0.4\sqrt{(1.8)(16.6)(1.57)\frac{(50)(1.1)}{(50)(1.1)}} = 2.74 \text{ in.}$$

(ANSI/AISC 358 [2005], 2.4.4-1)

$$t_{cf} \geq \frac{b_{bf}}{6} = 2.77 \text{ in.}$$ (ANSI/AISC 358 [2005], 2.4.4-2)

For the W36 × 395 column:

$$t_{cf} = 2.2in. < 2.74 \text{ in and } 2.77 \text{ in.}$$
$$\therefore \text{Continuity plates are required}$$

Using the requirements outlined in the design procedure, the thickness of the continuity plate (t_{bf}) should be greater than the thickness of the thinnest beam flange (if more than one beam frames into a column. Therefore, the projected contact area between the continuity plate and the column flange is calculated as:

$$A_{pb} = W_{pb-flange} * t_{cont-pl}$$

$$b_{cont-pl} = \frac{b_{cf} - t_{cw}}{2} = \frac{16.8 - 1.22}{2} = 7.8 \text{ in.}$$

$$W_{pb-flange} = b_{cont-pl} - ("k1_{col}" + 0.25in.) = 7.8 - (1.8125 + 0.25) = 5.73 \text{ in.}$$

$$t_{cont-pl} = 1.625 \text{ in.} \geq 1.57 \text{ in.} = t_{bf}$$

$$\therefore A_{pb} = 9.31 \text{ in}^2$$

The maximum contact area between the continuity plate and the column web is:

$$A_{pw} = t_{cont-pl}[d_c - 2t_{cf} - 2(k + 1.5in.)] = 1.625[38.4 - 2(2.2) - 2(3.44 + 1.5)] = 39.2 \text{ in.}^2$$

The four controlling cases described in the design procedure are as follows:

a) $\sum \phi F_y A_{pb} = (2)(0.9)(50)(9.31) = 838 \text{ kips}$ (ANSI/AISC 358 [2005], 2.4.4b (a))

b) $\phi_v V_{nw} = \phi(0.6) F_y A_{pw} = (1.0)(0.6)(50)(39.2) = 1{,}176 \text{ kips}$
(ANSI/AISC 358 [2005], 2.4.4b (b))

c) $\phi R_v = 1{,}603 \text{k } (no\,doubler),$ $\phi R_v = 3{,}331 \text{k } (doubler)$ (See Step 11 and ANSI/AISC 358 [2005], 2.4.4b (c))

d) $\sum \frac{\phi_d M_{pe}}{d_b - t_{bf}} = 2\left(\frac{1.0(65{,}450)}{37.1 - 1.57}\right) = 3{,}684 \text{ kips}$ (ANSI/AISC 358 [2005], 2.4.4b (d))

The smallest value is 838 kips, which will be used to design the welds between the continuity plate and the column web.

The minimum required double-sided fillet weld size to develop 838 kips is as follows:

$$D_{\min} = \frac{R_{ct-cw}}{2(1.392\,k/in.)[d_c - 2t_{cf} - 2(k+1.5in.)]}$$
$$= \frac{838\,kips}{2(1.392\,k/in.)[24.1in.]}$$
$$= 12.5(1/16^{th}\,weld) \cong 0.78in.$$

Where: $\phi R_{weld} = 0.75(0.6)(70ksi)(0.707)/16 = 1.392$ kip/in.

Use double-sided 7/16-inch fillet welds to connect the continuity plates to the column web. It will likely be more economical to use a partial joint penetration (PJP) weld that develops the strength of the plate or a CJP groove weld between the continuity plate and the column web—this can be established by the fabricator.

Step 11: Check column panel zone

$d_p = d_b - t_{bf} = 37.1 - 1.57 = 35.5$ in.

$M_f = 58{,}885$ kip-in. or $M_f = 57{,}518$ kip-in. (See Step 5)

$$R_u = \frac{\sum M_f}{d_p} = \frac{58{,}885 + 57{,}518}{35.5} = 3{,}279 \text{ kips}$$

The panel zone shear strength:

$$\phi R_v = \phi 0.6 F_{yc} d_c t_w \left[1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w}\right] \quad \text{(ANSI/AISC 360 [2005], J10-11)}$$
$$= 1.0(0.6)(50)(38.4)(1.22)\left[1 + \frac{3\,(16.8)\,(2.2)^2}{(37.1)(38.4)(1.22)}\right] = 1{,}603 < R_u$$

$\therefore$ Doubler plates are required.

where:

$\phi = 1.0$ per ANSI/AISC 341 (2005), Section 9.3a

$t_w = t_{cw} + t_{doubler\text{-}pl}$

Therefore, the W36 × 395 column panel zone strength (without doubler plates) is *not* adequate for the W36 × 282 beam.

The minimum panel zone thickness t_z is also checked per ANSI/AISC 341 (2005), Section 9.3b:

$$t_z \geq \left(d_z + w_z\right)/90 \qquad \text{(ANSI/AISC 341 [2005], 9-2)}$$

where:

d_z = panel zone depth between continuity plates = d_p
w_z = panel zone width between column flanges = d_{cf}-$2*t_{cf}$
$t_z = 1.22$ in. for W36 × 395(assume thickness of plate = thickness of column web)
$t_z = 1.22$ in. $\geq \left[(35.5) + \left(38.4 - 2(2.2)\right)\right]/90 = 0.77$ in. *OK*

Using ANSI/AISC 360 (2005}, equation J10-11, and solving for t_w, we can determine $t_{doubler\text{-}pl}$:

$$t_{doubler-pl} = \frac{R_u - \phi 0.6 F_{yc} \dfrac{3 b_{cf} t_{cf}^2}{d_b}}{\phi 0.6 F_{yc} d_c} - t_{cw} = \frac{3279 - 0.6(50)\left(\dfrac{3(16.8)(2.2)^2}{37.1}\right)}{0.6(50)(38.4)} - t_{cw} = 1.46 \text{ in.}$$

Using symmetric doubler plates (one on each side of the column web, each with a thickness of 3/4 inch) calculate $\phi R_v = 3{,}331$ kips $> R_u$. Detailing requirements for doubler plates are found in ANSI/AISC 341(2005), Section 9.3c. Note that doubler plates are expensive and difficult to install. A more cost-efficient option may be to increase the column member size, providing a thicker column web. However, the strength required for this section necessitates an extremely heavy column (W36 × 800). The engineer should consult a fabricator before changing from doubler plates to a heavier column or a column with higher F_y.

Step 12: Check column beam moment ratio

Check:

$$\sum M_{pc}* / \sum M_{pb}* > 1.0 \qquad \text{(ANSI/AISC 341 [2005], 9-3)}$$

The axial load on the column is taken from the computer model: P_{uc} = 142 kips

$$\sum M_{pc}* = \sum [Z_{xc}(F_{yc} - P_{uc} / A_g)]$$

$$= 2(1{,}710)(50 - \frac{142}{116}) = 166{,}813 \text{ kip/in.}^2$$

$$\sum M_{bc}* = \sum (C_{pr} R_y F_{yb} Z_{RBS} + M_v)$$

$$= 2(1.15)(1.1)(50)(771.6) + 33{,}782 = 131{,}390 \text{ kip/in.}^2$$

where $\sum M_v = (V_{RBS} + V'_{RBS})(S_h)$

$$= (419 + 363)(43.2) = 33{,}782 \text{ kip-in.}$$

$$\therefore \sum M_{pc}* / \sum M_{pb}* = 166{,}813 / 131{,}390 = 1.27 > 1.0 \ \ OK$$

Step 13: Evaluate lateral bracing of columns

From Step 12 of this design example:

$$\frac{\Sigma M^{*}_{pc}}{\Sigma M^{*}_{pb}} = 1.27 < 2.0$$

$\therefore$ Lateral bracing of the column flanges is required.

Satisfying the conditions outlined in the design procedure, the column flanges need to be laterally braced at the moment beam bottom flange only.

This bracing may be provided by perpendicular beams connected to full-depth stiffeners and column continuity plates. This requires that the perpendicular beams framing into the column have the same depth as the frame beams.

Another option to prevent warping or twisting of the column flange is to connect a single point brace at the frame beam bottom flange location. At the other end, the point brace may be connected to the top flange of the perpendicular beam or the slab interface (Figure 4-1). The design of the angle shown in Figure 4-1 is similar to the design of the lateral brace for the moment beam (Step 14). In most cases, the size of the angle used for column bracing will be the same size as the angle used for lateral bracing of the moment beam (for ease of fabrication and erection).

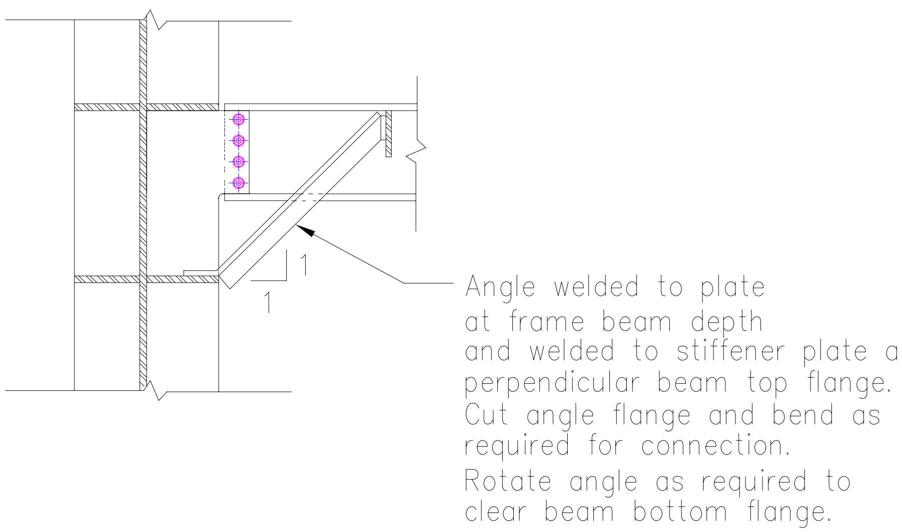


Figure 4-1: Column Bracing Detail

Step 14: Check beam lateral bracing requirements

All three cases discussed in the design procedure for lateral bracing follow below.

For Cases A, B, and C, determine maximum unbraced length per ANSI/AISC 341 (2005), 9.8:

$$L_{br} = \frac{0.086 r_{by} E}{F_{yb}} = \frac{0.086(3.8\,in.)(29{,}000\,ksi)}{50\,ksi} = 190\,in. = 15.8\ \text{ft}$$

Place minimum bracing at third points: $L_b = 28/3 = 9.33$ ft on center.

(Beam supports concrete structural slab per exception ANSI/AISC 341 [2005], 9.8.)

Case A: Nodal bracing with an angle near the plastic hinge location

For lateral brace strength requirement at the hinge location:

$$P_{br} = 0.02\frac{M_r C_d}{d_b - 2(t_{bf}/2)} = 0.02\frac{48{,}829(2.0)}{(37.1 - 1.57)} = 55\ \text{kips}$$

The length of the brace is assumed to be measured from the centerline of the W36 × 282 to the centerline of the adjacent gravity beam. Assuming a 12 foot, 6 inch beam spacing, the length of the brace is:

$$L_{brace} = \sqrt{(12.5\,ft)^2 + (d_b)^2} = 154.5\,in. = 12.9\ \text{ft}$$

From the AISC *Steel Construction Manual*, 13th edition (AISC 2005), we will try an *L*6*x*6*x*7/8 designed as an eccentrically loaded single angle with A_g = 9.75 in.^{2} and an unbraced length of 12.9 ft. The axial capacity of this element, per Table 4-11 is:

$$\phi R_n = 60.4\,kips > R_u = 55\ \text{kips}$$

The required stiffness is:

$$\beta_{br} = \frac{10 M_u C_d}{\phi L_b h_o} = \frac{10(48{,}829)(2.0)}{0.75(9.33 * 12)(35.5)} = 327\ \text{k/in.}$$

The brace stiffness can be calculated as:

$$k = \frac{A_g E}{L_{brace}}\cos^2(\theta)$$

$$\theta = \tan^{-1}\left(\frac{38.4\,in.}{12.9\,ft(12\,in./\,ft)}\right) = 13.94^\circ$$

$$k = \frac{9.75\,in.^2\,(29{,}000\,ksi)}{154.5\,in.}\cos^2(14.4^\circ) = 1{,}716\ \text{k/in.}$$

$$k > \beta_{br} \qquad OK$$

$L6x6x7/8$ kickers at 9.33 feet on center are adequate to brace the beam bottom flange. The kicker angles should be connected to beams (SMF and adjacent gravity framing) with fillet welds to stiffener plates between the beam flange and beam web.

Case B: Nodal bracing with a perpendicular beam

The lateral brace requirements for L_b, P_{br}, and β_{br} are identical to Case A.

a) Check for required torsional bracing:

1) $M_{br} = 0.024 M_r L_{pbm} / (nC_{bb} L_b) = 1{,}758$ kip-in.

Where $L_{pbm} = 28$ ft (336 in.)

$L_b = 9.33$ ft (112 in.)

$M_r = M_{pr} = 48{,}829$ kip-in.

$I_y = 1{,}200$ in.^{4}

$n = 2$

$C_{bb} = 1.0$ (Conservative to use 1.0)

$S_{xbrace} = M_{br}/(0.9F_y) = 1{,}758/(0.9(50ksi)) = 39.0$ in.^{3}

2) $$\beta_T = \frac{2.4 L_{pbm} M_r^2}{\phi(nEI_y C_{bb}^2)} = \frac{2.4(336)(48{,}829)^2}{0.75(2 \cdot 29{,}000 \cdot 1200 \cdot 1.0)} = 36{,}833 \text{ kip-in./rad}$$

3) $$\beta_{sec} = 3.3 \frac{E}{d_b - t_{bf}} \left[\frac{1.5(d_b - t_{bf}) t_{bw}^3}{12} + \frac{t_{stiff} b_{stiff}^3}{12} \right]$$

$$\beta_{sec} = 3.3 \left(\frac{29{,}000}{37.1 - 1.57} \right) \left[\frac{1.5 \cdot (37.1 - 1.57) \cdot (0.885)^3}{12} + \frac{(0.375) \cdot (7.86)^3}{12} \right]$$

$\beta_{sec} = 49{,}165$ kip-in./rad

For $b_{stiff} = (b_{bf} - t_{bw})/2 = 7.86$ in.

For $t_{stiff} = 0.375$ in (reasonable assumption—iterate if required)

4) $$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} = \frac{36{,}833}{\left(1 - \frac{36{,}833}{49{,}165}\right)} = 146{,}845 \text{ kip-in./rad}$$

Using a W16 × 31 beam as the bracing beam: $S_{xbrace} = 47.2$ in.^{3} and $I_{brace} = 375$ in.^{4}

For the W16 × 31 beam: $\beta_b = \dfrac{6EI_{brace}}{L_{brace}} = 194{,}196$ kip-in./rad > 146,845 kip-in./rad *OK*

Design weld for connection of beam to stiffeners:

$M_{br} = 1{,}758$ kip-in.

From the AISC *Steel Construction Manual*, 13th edition (AISC 2005), Table 8-8

Use a = 1.0 and k = 0.4:

C = 1.44, $P_u = M_{br}/l$ = 1,758/13.63 = 129 kips

$$D_{\min} = \frac{P_u}{\phi C C_1 l} = \frac{129}{0.75 \cdot 1.44 \cdot 1.0 \cdot 13.63} = 8.8(16th)$$

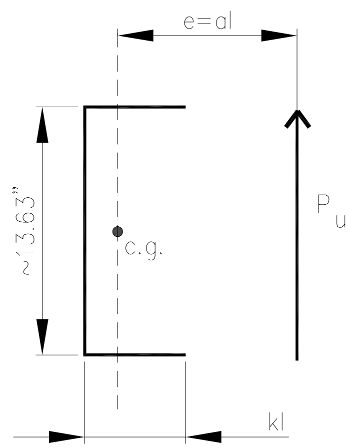


Figure 4-2: Free Body Diagram for Weld Between Beam and Plate

Therefore, a 5/8-inch fillet weld is required for the connection of the stiffeners to the beam, which will require the use of a plate with thickness 11/16 inch or greater—therefore a ¾-inch plate is required. However, the web and flange thickness of the W16 × 31 are 1/4 inch and 7/16 inch, respectively, not a recommended thickness for a 5/8-inch fillet weld based on heat input. A more efficient option could be to use a deeper bracing beam, lengthening the *l* dimension and reducing the minimum weld size and associated plate thickness. A W21 will allow a weld length of approximately 18 inches, resulting in a 5/16-inch fillet weld, which is more appropriate for the contemplated conditions (3/8-inch plate). Confirm an appropriate direction with the fabricator.

Case C: Relative bracing (flying beam condition)

Determine the brace force and stiffness if considered a nodal brace (for connection design):

$$P_{br} = \frac{0.02 M_r C_d}{h_o} = \frac{0.02 M_r C_d}{d_b - 2(t_{bf}/2)} = \frac{0.02 \cdot 48{,}829 \cdot 2.0}{37.1 - 1.57} = 55 \text{ kips}$$

$$\beta_{br} = \frac{10}{\phi L_b}(50 P_{br}) = \frac{10}{0.75(L_b)}(50)(55) = 109 \text{ kip-in./rad} \quad \text{for } L_b = 28 \text{ ft}$$

A welded plate connection will provide adequate strength and stiffness (Figure 4-3). The plate thickness, weld design, and associated details should be confirmed using ANSI/AISC 360 (2005). A sample connection detail is presented in Figure 4-3.

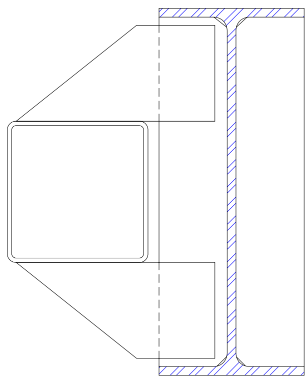


Figure 4-3: HSS to Beam Connection Schematic Detail

To design the relative brace:

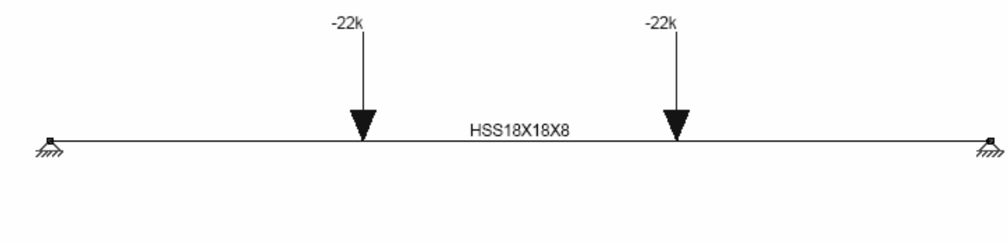
$$P_{br} = 0.008 \frac{M_r C_d}{h_o} = 0.008 \frac{48{,}829 \cdot 2.0}{37.1 - 1.57} = 22 \text{ kips}$$

$$\beta_{br} = \frac{4 M_r C_d}{\phi L_b h_o} = \frac{4 \cdot 48{,}829 \cdot 2.0}{0.75 \cdot 336 \cdot (37.1 - 1.57)} = 43.6 \text{ kip-in./rad}$$

At point of bracing: $\beta_{br} = P_{br} / \Delta_{rbr}$

Solve for Δ_{rbr} : $\Delta_{rbr} \le 0.504$ in. (Use this value to determine the required beam size.)

Try HSS18 × 18 × 1/2 bracing beam, connected to a W36 beam with a stiffener plate connection at third points (Figure 4-4). This HSS is connected to the gravity beams perpendicular to the moment beam with knife plates (pin connection). Various connection details will work for this condition as the fixity at the end of the HSS is not accounted for in this calculation.



Try Figure 4-4: HSS Model for Bracing Element

Model the condition above using an HSS18 × 18 × 1/2, $\Delta_{rbr} = 0.43$ in. at the point of bracing. *OK*

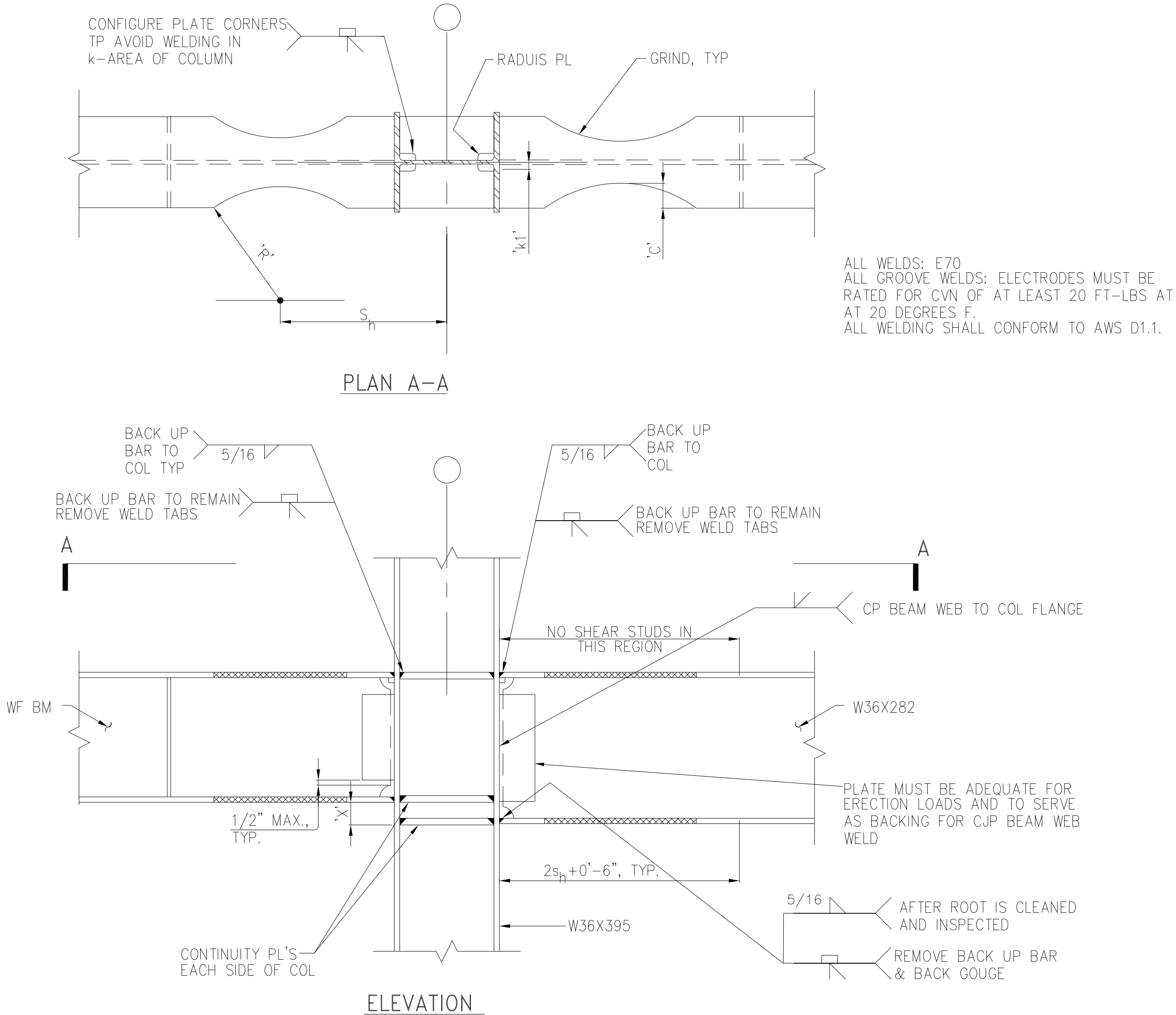


Figure 4-5: Schematic "General" Reduced Beam Section Connection Detail
(No Doubler Plate Shown)

5. Detailing Considerations

The fabrication and construction detailing requirements in ANSI/AISC 341 (2005) and ANSI/AISC 358 (2005) for reduced beam section (RBS) connections are summarized below.

- The RBS cut should be made by thermal cutting.
- The finished cut should have a maximum surface roughness of 500 microinches, avoiding nicks, gouges, and other discontinuities.
- All corners should be rounded to minimize notch effects, and cut edges should be ground in the direction of the flange length.
- Gouges and notches that occur in the thermally cut RBS surface may be repaired by grinding. If a sharp notch exists, the area shall be inspected by the Magnetic Particle Testing (MT) method after grinding to ensure that the entire depth of notch has been removed.
- Gouges and notches that exceed 1/4 inch in depth but that do not exceed 1/2 inch in depth may be repaired by welding. Notches and gouges exceeding 1/2 inch in depth shall be repaired only with a method approved by the engineer of record.
- Bolt holes in beam webs, as permitted in ANSI/AISC 358 (2005), shall be permitted.
- Welded shear studs should be omitted within the protected zone if a concrete fill over composite metal deck is present.
- Parameters for prequalified welded joints are presented in ANSI/AISC 358 (2005), Chapter 3.
- Requirements for backing and weld tabs are described in ANSI/AISC 358 (2005), Chapter 3.

In addition, ANSI/AISC 341 (2005), AWS D1.1 (2004), and AWS D1.8 (2005) dictate certain parameters associated with detailing of plates, stiffeners, welds, and fabricated elements. Specific details, processes, and materials related to weld access holes, weld preparation, and fabrication aids should strictly adhere to dimensions, requirements, and conditions as published in AISC standards and AWS standards. Specific details can be developed by the designer or referenced for use by the fabricator. In any case, it is important to ensure that specific connection characteristics adhere to published values.

6. Quality Assurance and Quality Control

Quality assurance and quality control requirements are outlined in ANSI/AISC 358 (2005), Section 3.7, referring to ANSI/AISC 341 (2005), Appendix Q, and AWS D1.8 (2005), much of which supplants the recommendations and requirements promulgated in the AISC standards as related to welding. The designer should utilize these guidelines to ensure the proper selection and handling of materials and shop and field fabrication of the RBS connections.

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