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Design of Special Concentric Braced Frames

(With Comments on Ordinary Concentric Braced Frames)

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NOTATIONS

Where possible, the reference source for the notation is indicated in brackets.

Notation	Definition
A	Area of cross section (in^2)
A_e	Effective area (in^2)[AISC, LRFD Manual]
A_g	Gross area (in^2) [AISC, LRFD]
A_{gt}	Gross area subject to tension (in^2) (block shear failure check) [AISC, LRFD]
A_{gv}	Gross area subject to shear (in^2)(block shear failure check)
A^n	Net area (in^2)
A_w	Whitmore area (in^2)
B	Stress reduction factor used for ASD design of OCBF [UBC, 1997]
C_a	Seismic coefficient in table 16-Q [UBC, 1997]
C_t	Numerical coefficient in section 1630.2.2 [UBC, 1997]

Notation	Definition
C_v	Seismic coefficient from 1997 UBC, as set forth in table 16-R [UBC, 1997]
D	Dead load [UBC, 1997]
E	Earthquake load [UBC, 1997]
E	Modulus of elasticity (ksi)
E_h	Earthquake load (horizontal) [UBC, 1997]
E_S	Steel modulus of elasticity = 29,000,000
F_a	Allowable stress in compression [AISC ASD Manual]
F_{cr}	Critical stress (ksi) [AISC LRFD]
F_p	Design seismic forces in part of the structure [UBC, 1997]
F_t	That portion of the building base shear, V, considered concentrated at the top of the structure (in addition to Fn)- whiplash effect check (1997 UBC)
F_i, F_n, F_x	Design seismic force applied to level i, level n, level x respectively
F_u	Specified minimum steel tensile stress (ksi) [AISC]
F_y	Specified minimum steel yield strength (ksi)[AISC]
H	Height of object or building
H_{story}	Height of that particular floor level
I	Moment of inertia (in^4)
I	Importance factor given in table 16-K [UBC, 1997]
K	Effective length factor for compression members
L	Unbraced length of compression or bracing member [AISC]
L	Live load [UBC, 1997]
L_{fg}	Length of free edge of gusset plate (inches)
L_g	Gusset plate length (inches)
M	Bending moment
M_u	Required bending strength of a member
N	Number of stories in a building [UBC, 1997]
N_a	Near-source factor (also called the near field factor) [UBC, 1997, tables 16-S and 16-U]
$N_{stories}$	Number of floor levels (stories) in building
N_v	Near-source factor used in determining C_v [UBC, 1997]
P_l	Axial live load force on framing member
P_d	Axial dead load force on framing member
P_e	Axial seismic force on framing member
P_n	Nominal axial strength (tension or compression in kips) [AISC, LRFD]
P_{rl}	Roof axial live load force
P_u	Required axial strength of a member [AISC, LRFD]
P_{uc}	Member axial load capacity (Compression) using strength level design
P_{ut}	Member axial load capacity (tension) using strength level design
P_{cr}	Critical axial strength of a member in compression
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces (kips) [AISC *Seismic Provisions*, 2002]
R	Numerical coefficient representative of the inherent

Notation	Definition
	overstrength and global ductility capacity of lateral force resisting system - table 16-N or 16-P [UBC, 1997]
R_f	Ratio of the expected steel tensile strength to the minimum specified tensile strength F_u [currently being researched by AISC]
R_n	Nominal strength [AISC]
R_w	Numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral-force-resisting systems [UBC, 1997, section 1628]
R_y	Ratio of the expected steel yield strength to the minimum specified yield strength F_y [AISC *Seismic Provisions*, 2002]
S_D	Soil profile type D [UBC, 1997, table 16-J]
T	Elastic fundamental period of vibration of the structure in direction under consideration (seconds) [UBC, 1997] T is also a dimension used in gusset plate geometry.
T_a	The elastic fundamental period of vibration, in seconds, of the structure as determined per Method A [UBC, 1997, 1630.2.2]
T_b	The elastic fundamental period of vibration, in seconds, of the structure as determined per Method B [UBC, 1997, 1630.2.2]
U	Shear lag reduction factor used to calculate A_e [AISC, LRFD]
V	Shear force or base shear [UBC, 1997]
V_{max}	The maximum total design lateral force or shear at the structure's base as determined from base shear equations [UBC, 1997, 1630.2]
V_{min}	The minimum total design lateral force or shear at the structure's base as determined from base shear equations [UBC, 1997, 1630.2]
V_x	Total design lateral force or shear at the base determined from formulas in the building x direction [UBC, 1997, 1630.2]
V_y	Total design lateral force or shear at the base determined from formulas in the building y direction [UBC, 1997, 1630.2]
W	Total seismic dead load [UBC, 1997]
W_p	Seismic weight of an element or component [UBC, 1997]
Z	Plastic section modulus (in^3) [AISC]
Z	Seismic zone factor [UBC, 1997]
b	Width of compression element as defined in LRFD specs. [AISC, LRFD]
b_f	Column flange width
h_a	Building height (in feet) used in calculating building period "T"
h_x	Height in feet above the base to level "x" [UBC, 1997, section 1628]
r	Governing radius of gyration

Notation	Definition
r_i	The individual elements story shear ratio used in determing the redundancy/reliability factor "ρ" [UBC, 1997]
r_{max}	The maximum ratio used in determining the redundancy/reliability factor "ρ" [UBC, 1997]
rx/ry	Ratio of radius of gyration of member x axis to y axis
t	Thickness of element (in.) [AISC]
t_f	Thickness of column flange
w_x	Total weight associated with floor level "x"
α	Dimension along beam used in Uniform Force Method for determining gusset plate forces [AISC, LRFD]
β	Dimension along column used in Uniform Force Method for determining gusset plate forces [AISC, LRFD]
Δ_m	Maximum inelastic response displacement [UBC, 1997]
Δ_{mroof}	Maximum Inelastic Response Displacement (at roof)
Δ_s	Design level response displacement [UBC, 1997]
Ω_0	Horizontal seismic overstrength factor [1997 UBC and AISC *Seismic Provisions*, 2002]
φ	Resistance factor [AISC, LRFD]
ρ	Redundancy/reliability factor [UBC, 1997]
ρ_x	Redundancy/reliability value (Rho) in building x direction [UBC, 1997, section 1630.1.1]
ρ_y	Redundancy/reliability value (Rho) in building y direction [UBC, 1997, section 1630.1.1]
λ_c	Column slenderness parameter [AISC, LRFD]
λ_{ps}	Limiting slenderness parameter for compact elements [AISC *Seismic Provisions* 2002]

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PART A: GENERAL DESIGN

1.0 INTRODUCTION

1.1 Preface

Steel-braced frames are recognized as a very efficient and economical system for resisting lateral forces and minimizing building drifts. Braced frame systems are efficient because framing members resist primarily axial loads with little or no bending in the members until the compression braces in the system buckle. Framing members are initially sized based upon the following criteria:

- Sufficient stiffness (member cross-sectional shape and area) to satisfy code drift requirements.
- Adequate member strength to resist both compression and tensile axial forces.

After selecting initial framing member sizes, minimum building code design requirements for framing members and connections must then be checked and satisfied.

- Adequate beam strength to resist induced bending forces as a result of V (chevron) brace buckling in Special Concentric Braced Frames (SCBF).
- Code slenderness requirements for brace members.

Booklets on the subject of braced-frame design previously published by the Structural Steel Education Council (SSEC) Technical Information and Product Service (TIPS) and the Structural Engineers Association of California (SEAOC) include:

- *Steel TIPS*, "Seismic Design Practice for Steel Buildings" (June 1988)
- *Steel TIPS*, "Seismic Design of Special Concentrically Braced Steel Frames," (November 1995)

- *Steel TIPS*, "Seismic Behavior and Design of Gusset Plates" (December 1998)

- SEAOC *Seismic Design Manual*, Volume III, Concrete and Steel Building Design Examples (November 2000 – Uniform Building Code (UBC), 1997)

- SEAOC *Seismic Design Manual*, Volume III, Concrete and Steel Building Design Examples (Fall 2003 – International Building Code (IBC), 2000)

1.2 Purpose of This *Steel TIPS*

The primary focus of this edition of *Steel TIPS* is to present an update on the selection and design of SCBF bracing members and connections, including a discussion of the currently applicable codes and guidelines. Various types of bracing member connection designs and fabrication are discussed. A brief comparison of Ordinary Concentric Braced Frames (OCBF) and SCBF is contained in the next section, but the focus of this document is on SCBF since in areas of high seismicity, SCBF should be used instead of OCBF. However, OCBF can be used under certain limited conditions in areas of high seismicity.

A design example for a low-rise building using SCBF in both directions is presented in Part B. This design example follows the Load and Resistance Factor Design (LRFD) method rather than the Allowable Stress Design (ASD) method since the seismic loads derived from the current model codes (such as the 1997 UBC and all editions of the IBC) are based on strength rather than allowable loads. This is discussed in more detail in section 1.3.

1.3 Current Building Codes

The three model building code agencies in the United States - the International Conference of Building Officials (ICBO), the Standard Building Code (SBC), and the Building Officials and Code Administrators (BOCA) – have merged into a single model building code agency, the International Code Council (ICC). The ICC has begun publishing new model building codes, including the International Building Code (IBC), which contains the structural design requirements. The first edition of the IBC was published in 2000 and the ICC has adopted the same three-year code development cycle between new model building code editions. The 2000 IBC adopts the 1997 AISC *Seismic Provisions for Structural Steel Buildings* (including Supplements 1 & 2) by reference. The 2003 edition of the IBC that was just recently published has adopted the current 2002 AISC *Seismic Provisions* by reference. In other words, the SCBF and OCBF design requirements are no longer reprinted in the IBC, as they were in the 1997 UBC.

The AISC *Seismic Provisions* are considered the state of the art for structural steel design and should be used when possible. Besides the IBC, both NEHRP and the ASCE-7 design standard, which are used in the development of model building codes, incorporate by direct reference the AISC *Seismic Provisions* for SCBF and OCBF. The 1997 UBC is an outdated code and is partially based on the 1997 version of the AISC *Seismic Provisions*.

There are significant differences between the older 1997 and the current 2002 AISC *Seismic Provisions* and the 1997 UBC, including the following:

- The 1997 and 2002 AISC *Seismic Provisions* require the use of φ factors in ASD design that are less than 1.0. The φ factors are not stated in the 1997 UBC and are therefore inferred to be 1.0.

- The 1997 and 2002 AISC *Seismic Provisions* require the use of R_y (ratio of expected yield strength to the minimum specified yield strength modifier), which is not in the 1997 UBC.

- The 2002 AISC *Seismic Provisions* has simplified the design requirements for OCBF, making the 1997 UBC design requirements more cumbersome.

1.3.1 The California Building Code

The 1997 UBC is used in this publication since it is the basis for the current 2001 California State Building Code. California, as of July 2003, has chosen to adopt the National Fire Protection Agency (NFPA) 5000 model building code instead of the IBC model building code despite wide support for adopting the IBC within the structural engineering community and other state engineering agencies. This decision may be overturned at a later date. The NFPA 5000 model code, as adopted by California, is tentatively scheduled to become effective in January 2006 and adopted by all cities and counties by July 2006 (along with any local amendments). Both the IBC and NFPA 5000 model code reference the 2002 AISC *Seismic Provisions* and ASCE 7-02 document for seismic design provisions, so there is no expected impact on the design of brace frames, other than possibly more stringent amendments that might be adopted by the State of California, or local jurisdictions.

The discussion of the design and detailing of SCBF framing members and connections in this *Steel TIPS* is based upon 2002 AISC *Seismic Provisions* for SCBF frame design and detailing. Design loads were determined based upon the 1997 UBC using LRFD Load Combinations. The 1997 UBC was used instead of the 2000 IBC, since the 1997 UBC remains the adopted model building code for California until at least 2006. The LRFD method was chosen instead of the ASD method, since the seismic forces generated from current codes (UBC, IBC and

ASCE-7) are based on strength design. Also, the requirements for SCBF in the 2002 AISC *Seismic Provisions* are based on strength and the ability of SCBF to provide ductility (inelastic action). The LRFD method is being emphasized by AISC even though the ASD design method is still being used in many design offices. In the authors' opinion, ASD is an outdated method. The 1997 UBC, IBC and ASCE-7 still allow ASD as an alternate method, but converting LRFD load combination equations to ASD load combinations is awkward and not straightforward.

Note that the 1997 UBC refers to OCBF as Concentric Braced Frames (CBF). The IBC and AISC *Seismic Provisions* and this *Steel TIPS* use the acronyms OCBF and SCBF. Designers are also reminded that in jurisdictions which have adopted the 1997 UBC (example: California), they cannot necessarily substitute the current AISC *Seismic Provisions* for all SCBF and OCBF design requirements, since the 1997 UBC still has some provisions which are more restrictive (example: b/t ratios for OCBF). This includes local jurisdictions in California, which have adopted the AISC *Seismic Provisions* in lieu of chapter 22 in the 1997 UBC. Preferably the designer will not blindly use the 1997 UBC, but will incorporate the latest edition of the AISC *Seismic Provisions* as applicable.

The Steel Braced Frame and related subsections of chapter 22 (General) and chapter 22A (Division of State Architect [DSA], Office of Statewide Health Planning and Development [OSHPD]) of the 2001 California State Building Code are the same as the 1997 UBC with the following known exceptions:

1. General Braced Frame Design (Chapter 22)

   a. State: no known state code changes to Braced Frames
   b. Local jurisdictions: the following known changes:

      2002 City of Los Angeles Building Code, City of Santa Monica Municipal Building Code:

      Adopted AISC 1997 *Seismic Provisions*, including Supplement #2
         OCBF modified: R = 5.0, Ω_o = 2.0
         OCBF Height Restriction = 35 feet

2. DSA/OSHPD Braced Frame Design (Chapter 22A)

   Section 2211A: Has adopted the AISC *Seismic Provisions* for Structural Steel Buildings, April 15, 1997, including Supplement No. 1, dated February 15, 1999

   Section 2213A.4.1: Requires weld filler material have a minimum Charpy V-notch toughness of 20 foot-pounds at minus 20 degrees F.

Section 2213A.8.5: OSHPD has not adopted this *One- and Two- Story Building Section* for OCBF (Note: OSHPD has not adopted the use of OCBF basic structural system (chapter 16A, table 16A-N, footnote 10) and requires braced frames to be SCBF.

The authors strongly recommend that designers check with local jurisdictions for any local building code amendments prior to starting their designs.

2.0 GENERAL DESIGN INFORMATION

2.1 OCBF Compared to SCBF

The 1997 UBC essentially divides the design of OCBF and SCBF systems into five parts, which are described in the sections below:

2.1.1 Brace Design

2.1.2 Brace Connection

2.1.3 Brace Configuration

2.1.4 One- and Two-Story Buildings (OCBF Only)

2.1.5 Column Requirements (SCBF Only)

2.1.6 Non-Building Structures

The 1997 UBC design requirements for OCBF are provided in section 2211.4.9 of the *Manual of Steel Construction: Load & Resistance Factor Design*, 3rd Ed., American Institute of Steel Construction, Chicago, 2001 (LRFD Manual) and section 2213.8 of the *Manual of Steel Construction: Allowable Stress Design*, 9th Ed., American Institute of Steel Construction, Chicago, 1989 (ASD Manual). The design requirements for SCBF are provided in sections 2210.10 (LRFD Manual) and 2213.9 (ASD Manual).

2.1.1 Brace Design

There is no difference in performance between an OCBF and an SCBF until the compression brace buckles. What separates the OCBF from the SCBF are the restrictions on brace frame members and the detailing of the connections.

OCBF depend on limited brace buckling and do not have the ductile detailing of SCBF; therefore restrictions are placed upon the slenderness ratio (Kl/r) for the brace members. In the UBC for ASD design the allowable axial load is modified by the "B" value. As the brace member becomes longer, the axial load required to cause buckling becomes less. To prevent premature buckling, the "B" values become smaller as the brace becomes longer, reducing the allowable load F_a. This reduction in the braces' allowable axial compression design capacity is to help compensate for when actual seismic loads exceed design loads. For LRFD design, the design strength of a bracing member in compression shall not exceed $0.8_{\varphi}cP_n$, whereas for SCBF the design strength shall not exceed $\varphi_c P_n$.

The SCBF allows for buckling of the bracing members and is a more ductile system than OCBF; therefore, the codes are not as restrictive as the OCBF regarding the brace slenderness ratio. Because buckling is allowed in SCBF, the critical buckling axial load is not reduced. Brace buckling is allowed because special gusset plate detailing is required for both in-plane and out-of-plane brace buckling design, depending upon brace buckling mode selected. When built-up members are used for braces, the stitching l/r ratios are more restrictive for the SCBF than the OCBF. If it can be shown that SCBF braces can buckle without causing shear in the stitches, then the l/r ratios are the same for both the OCBF and SCBF.

The framing width-thickness requirements (b/t ratios) in the 1997 UBC are now the same for both the OCBF and SCBF. The reason for the change was to prevent localized premature buckling of a portion of the brace prior to overall axial buckling. Previously the OCBF was not as restrictive on the b/t ratios, but it was felt that localized buckling could occur prior to axial buckling in the OCBF, thus requiring more stringent b/t ratios. The more stringent b/t ratios also increase the expected post-buckling life of the steel after initial buckling.

The AISC 2002 *Seismic Provisions* have removed the b/t ratio requirements for OCBF by requiring higher design forces (Ω_o) for all framing members in the braced frame, but the b/t ratio requirements must still be complied with when using the 1997 UBC (example: California State Building Code). In the authors' opinion, the designer should give consideration to still also using the more stringent SCBF b/t ratios for OCBF.

2.1.2 Brace Connections

The connection design force requirements are the same for both the OCBF and SCBF, and must meet the least of the following forces:

1. Axial tensile capacity of brace, determined as $R_yF_yA_g$.

2. (Seismic Force Overstrength Factor Ω_o)x(brace seismic force + gravity loads)

(Note that the AISC 2002 *Seismic Provisions* omit this load combination for both OCBF and SCBF frames. The design engineer should give serious consideration to omitting this design force combination when using the 1997 UBC. Also note that the AISC *Seismic Provisions* refers to Ω_o as the "Overstrength Factor" and the UBC refers to Ω_o as the "Seismic Force Amplification Factor" to account for the structural overstrength.)

3. Maximum force that can be transferred to the brace by the lateral force resisting system.

Design of the brace connection also requires checking net area proportions (the effective net area to the gross area ratio) and tension shear lag effects (U) of the brace per code requirements.

The SCBF is only more restrictive regarding gusset plate buckling, which is not considered with the OCBF. The SCBF gusset plate must be designed for compression forces and detailed for possible out-of-plane bending if the brace is expected to buckle out-of-plane. The gusset plate for the OCBF needs to be checked for compression buckling as well, but direction of brace buckling is not considered.

2.1.3 Bracing Configurations

The following 1997 UBC restrictions for OCBF and SCBF are based on the braced bay configuration:

1. The 1997 UBC requires that OCBF inverted V (chevron braces) and V braces be designed for 1.5 times the prescribed seismic forces, which is not required in the SCBF. The AISC *Seismic Provisions* defers to the applicable building code for loads. (See figures 2-1D and 2-1E).

2. SCBF V or inverted V brace beams must be designed for unbalanced brace forces prescribed in the code when one brace buckles. The SCBF beams at the penthouse or roof level are exempt from this requirement.

3. SCBF V or inverted V brace beams must have their top and bottom flanges either directly or indirectly laterally braced at the point of the brace intersection. Though not explicitly stated in the building UBC code, this should apply to OCBF as well.

4. K bracing is prohibited in SCBF buildings and limited to one- and two-story buildings for OCBF. The authors do not recommend the use of K bracing for OCBF in moderate and high seismic zones (see figure 2-1C).

5. The 1997 UBC permits the use of non-concentric bracing connections (where the individual member centerlines do not all converge to one point at the connection) for both SCBF and OCBF provided the eccentricity is accounted for in the design (including all secondary moments). The UBC further requires that the worklines intersect within the width of the brace frame members (column depth, beam depth) for OCBF. The UBC gives no explicit limits on eccentricities for SCBF but, in the authors' opinion, the worklines should intersect within the width of the brace frame member as for OCBF. Interestingly, the AISC *Seismic Provisions* and the IBC have no requirements regarding non-concentric bracing connections. In the authors' opinion, the UBC limitations should be adhered to. There can be an advantage in terms of gusset plate size to using non-concentric connections. See section 6.2.5 for further discussion of non-concentric bracing connections and the Uniform Force Method.

2.1.4 One- and Two-Story Buildings (OCBF only)

OCBF not meeting the UBC prescriptive design requirements can still be used in buildings as long as the building is limited to two stories in height or is a roof structure. The penalty is that the brace design force must be increased by .4R ("R" is the Response Modification Factor in the UBC in table 16N), which is Ω_o listed in UBC table 16N.

The 2002 AISC *Seismic Provisions* have modified this so that all braced frame members (brace, column and beams) are to be designed for Ω_o times the design seismic force in these members. Brace connections shall be designed for $A_g F_y R_y$. There are some revised restrictions on V (chevron) braced frames. Building system height limits for OCBF have also been reduced to 35 feet in the IBC 2003. When designing the OCBF connections for the tensile capacity of the brace, the gusset plates will now be much larger, approaching the size of the SCBF gusset plates, in order to transfer the brace vertical and horizontal component forces to the beam/column connection.

The designer should be aware of the following during the initial design phase for a one- or two-story building with an OCBF system. If one particular brace in all the brace frame bays does not pass all the code requirements, and the designer then decides to use the one/two story exception, then all the brace frames in the building must be rechecked for the one/two story building criteria now including Ω_o.

It is quite likely that some or all of the other brace members that previously passed OCBF requirements, will have to be increased in size to satisfy the increased axial loads as a result of applying Ω_o to the seismic design forces. The one/two story exception shall not be applied to just the single brace frame bay that failed by itself. All brace frames in the building system should be designed for a similar force level, whether it is based upon the building system factor of R=6.4 (SCBF), R=5.6 or 5.0 (OCBF), or designing the brace frame members to include Ω_o when using the one/two story exception.

2.1.5 Brace Bay Columns (SCBF Only)

Because flexure is expected to occur in the SCBF, the SCBF columns, in addition to the OCBF design requirements, must be detailed for the column splice to occur in the middle one-third clear height between floors and the splice must be capable of developing 50 percent of the column moment capacity of the smaller column at the splice. The column splice must also develop the nominal shear strength of the smaller column member. This will likely require complete penetration welds of the upper column flanges and web to the lower column below the splice.

2.1.6 Non-Building Structures, Building Appendages (Rooftop Structures) and Discontinuous Systems

2.1.6.1 Non-Building Structures and Appendages

The seismic lateral forces for building appendages such as mezzanines, rooftop platforms, stair/elevator penthouses and equipment penthouses are typically derived from section 1632 of the 1997 UBC. Section 1632 is also applicable to non-building structures. Typically OCBF are used to brace these appendages since SCBF are not usually applicable.

When the seismic design forces for an OCBF are derived from section 1632.1 of the 1997 UBC using equation 32-1 ($F_p=4.0C_aI_pW_p$) or equation 32-2 ($F_p=(a_pC_aI_p/R_p)(1+3h_x/h_r)W_p$), the authors believe the building code should not require the brace connections to be designed for the tensile capacity of the brace or that Ω_o should be applied to the member and connection design as is currently being interpreted. The seismic design force F_p derived from these equations is significantly greater than those loads used for the main building system, especially when a_p is greater than 1.0 in equation 32-2.

Example: Building Seismic Zone 4, more than 5 km from Type B fault, Soil = S_D, I= 1.0, C_a= 0.44Na = 0.44(1.0) = 0.44

$V = (2.5C_aI/R)\ W = [2.5(0.44)(1)/5.6]\ W = 0.196\ W$

$$F_p \text{ (roof)} = (a_pC_aI_p/R_p)(1+3h_x/h_r)\ Wp$$
$$= [1.0(0.44)(1)/3][1+3(1/1)]$$
$$= 0.587\ W_p$$

$$F_p\text{(grade)} = .7C_aI_pW_p \text{ (min. design force governs at grade)}$$
$$= [0.7(0.44)(1)]W_p = 0.308\ W_p$$

The authors feel that to include omega in the OCBF member and connection design criteria of the 1997 UBC and 2002 AISC Seismic Design Provisions is excessive when seismic forces are derived using F_p. F_p represents the expected amplified seismic design force based upon the appendage's height in the building as determined from 1997 UBC section 1632.2, equation 32-1.

Each of the building lateral force resisting systems listed in the building codes assume a certain level of system ductility, incorporated by the building system factor (R). The seismic design forces in critical elements of the building's lateral force resisting system are then amplified by Ω_o to make sure they remain essentially elastic during the earthquake. Applying the amplification factor Ω_o to F_p seismic design forces would appear overly conservative since F_p already considers the amplification of seismic forces based upon the appendage's height in the building.

If the current building code interpretation is changed so when designing brace frame members and connections F_p design forces are not amplified by Ω_o, the designer should use some judgment when designing connections based upon using the F_p equation, since rooftop structures will be designed for much greater loads, whereas building appendages near grade well have much smaller design loads. When the building appendage is near grade, the designer may want to consider designing the connection for a larger force than that derived from the F_p equation since the design force may be less than the design force that would be determined from using the main building system factor R and Ω_o.

2.1.6.2 Discontinuous Systems

Often the framing members for these building appendages are part of a discontinuous lateral resisting system being supported by other framing members, which per section 1630.8.2 requires these framing members to be evaluated using the load combinations of section 1612.4 which include $E_M = \Omega_o E_h$. The 1997 UBC section 1630.1 defines E_h as equal to either V or F_p, which means F_p would be amplified by Ω_o when using $E_M = \Omega_o E_h$. The 1997 UBC section 1632.2 states that F_p shall be used with the load combination equations from section 1612.2 and 1612.3, but nothing is said about using section 1612.4

in which load combinations include E_M. Similar wording occurs in the 2000 IBC for the load combinations, F_p and E_M.

As discussed in section 2.1.6.1 above, further amplifying the F_p seismic design forces by Ω_o seems excessive. The designer is reminded that when evaluating the support members of a discontinuous system for an appendage, F_p is to be calculated at strength level, so if F_p was reduced for ASD, the F_p design forces must be amplified by 1.4 to convert back to strength level.

2.2 Bracing Layout Schemes

2.2.1 Lateral Force Considerations

The design seismic forces for any building using braced frame systems are dependent upon the layout of the frames and the number of bays that are braced. The building base shear V is dependant upon the reliability/redundancy factor "ρ" (in the UBC and IBC codes), which is calculated based upon the maximum horizontal force component in any single brace divided by the total story shear at that floor level. The maximum element-story shear ratio r_{max} is defined as the largest of the element-story shear ratios, r_i, which occurs in any of the story levels at or below the two-thirds height level of the building.

Buildings having a sufficient number of symmetrically distributed braced frames at each floor level will not be penalized by the reliability/redundancy factor. Buildings lacking a sufficient number of bracing members or having a poor frame layout will be penalized by the reliability/redundancy factor with up to a 50 percent maximum increase in base shear V. The redundancy factor "ρ" is calculated for both building directions.

A quick way to determine the preliminary minimum number of braces required at each floor level to satisfy redundancy requirements is to use the equation shown below. This is simply a reworking of the UBC redundancy penalty equation and assumes a symmetrically braced bay layout where all braces at a given floor level have approximately the same horizontal force component. The assumptions are that ρ = 1.0 and the force in the individual brace is directly proportionate to the number of braces.

$\rho = 2 - 20/((r_{max})\sqrt{A_b})$ (1997 UBC equation 30-3)

$1 = 20/((r_{max})\sqrt{A_b})$

$r_{max} = 20/\sqrt{A_b}$

Minimum Number of braces = $1/r_{max} = \sqrt{A_b}/20) = 0.05\sqrt{A_b}$

A_b = floor area of floor level under consideration (in square feet)

Again, if the brace layout is not symmetrical, or the number of braced bays is minimized, then there likely will be a design penalty for lack of redundancy in the building.

2.2.2 Overturning

The overturning design forces associated with braced-frame systems are usually greater than moment-frame systems because of the greater frame stiffness. This affects the building period "T" (which is shorter), and the lateral system coefficients "R" which are lower when calculating the building base shear V. Generally this results in a higher base shear than for more flexible systems.

UBC: Braced frames: R = 5.6 (OCBF) & 6.4 (SCBF)

IBC: Braced frames: R = 4.0 (OCBF) & 6.0 (SCBF)

This has a significant impact on the foundation design. The bracing connection to the footings must be of sufficient strength to positively transfer shear and overturning forces and prevent premature failure. Because of the high loads associated with braced frames, shear lugs welded to the underside of the column base plate, rebar welded to the base plate and special uplift anchors may be required to transfer these loads to the foundation.

Braced frames with an even number of bays (two bays, four bays, etc.) having equal bay widths (30 feet, 30 feet) represent a special case; typically the end columns take all the seismic uplift while the center column may take very little, if any, uplift. However, the interior columns should have uplift capacities similar to the end columns to provide redundancy should a first- or second-floor brace buckle causing redistribution of the uplift forces to the interior braced frame columns (see figure 2-1I).

When evaluating a buckled brace, 30 percent of φP_n is used. This is caused by load reversals during an earthquake; the buckled brace will straighten out, but will never fully return to the original straightness. Since the brace has previously buckled in compression, it will have significantly less compression axial capacity during subsequent tension-compression cycles, especially if there has been any inelastic damage, and will now buckle at a lower force level. Thirty percent of φP_n is a reasonable residual compression capacity and is supported by research done in Canada (Tremblay, *Engineering Journal*, AISC, 2001). It would be conservative to omit the buckled brace for load redistribution. However, this results in larger brace frame uplift forces and unbalanced beam bending forces. It is beneficial and still conservative to use 30 percent of φP_n for the compression capacity of the brace.

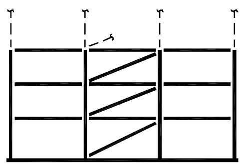
2.2.3 Bracing Configurations

Because of both increased overturning forces and the redundancy factor, the sizing of brace members can be greatly influenced based upon the number and distribution of braced frames within the building. The more braced frames and brace elements that are provided, the lower the brace force and resulting brace member sizes. Shown in figure 2-1G is a bracing configuration that can be utilized to help reduce overturning forces.

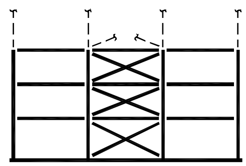
2.3 Buckling Mode of Bracing

The performance of the bracing system is based on the predicted mode of brace buckling, either in-plane or out-of-plane buckling. The buckling mode also impacts the design and detailing of the connections. SCBF have specific building code design requirements to ensure ductility of the SCBF connection when the brace buckles that are not required for OCBF. OCBF are designed for larger axial forces than SCBF to delay the onset of brace buckling, thereby reducing the ductility requirements for the OCBF connection. But if the seismic forces are actually large enough to buckle the OCBF brace, the OCBF connection may lack the ductility and detailing to resist the brace-bending moments induced into the gusset plate leading to connection failure, especially if brace buckling is in the out-of-plane direction. The lack of OCBF connection ductility is the reason for the restrictions limiting the use of OCBF in high seismic regions.

In-plane buckling of the brace may be the preferred mode of buckling rather than out-of-plane buckling, since it usually allows for greater energy dissipation by the bracing system as the frame attempts to deform in-plane. The reason for this is that when the brace buckles in-plane, it is buckling about the strong axis of the gusset plate. This forces a plastic hinge to form in the brace immediately adjacent to the gusset plate. The formation of these hinges in the brace ends makes a significant contribution to the energy dissipation potential of the frame.

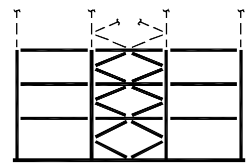


Single Diagonal
(Include Ω for design)
Figure 2-1A

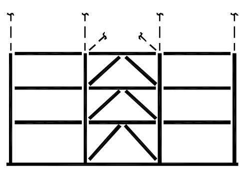


"X" Brace

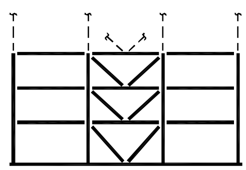
Figure 2-1B



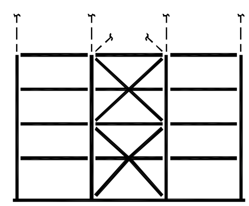
"K" Brace
(Do not use)
Figure 2-1C



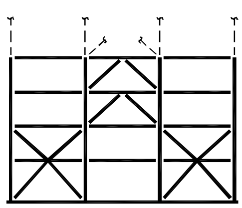
Inverted "V"
(Chevron Bracing)
Figure 2-1D



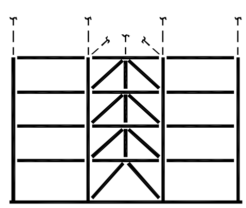
"V" Bracing
Figure 2-1E



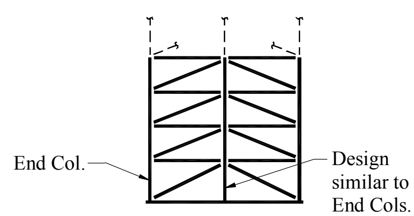
Two Story "X" Bracing
Figure 2-1F



Bracing Arraingement to
Spread Out Overturning
Figure 2-1G



Chevron Bracing
(With zipper column)
(No column at 1st floor)
Figure 2-1H



B.F. With Even
Number of Bays
Figure 2-1 I

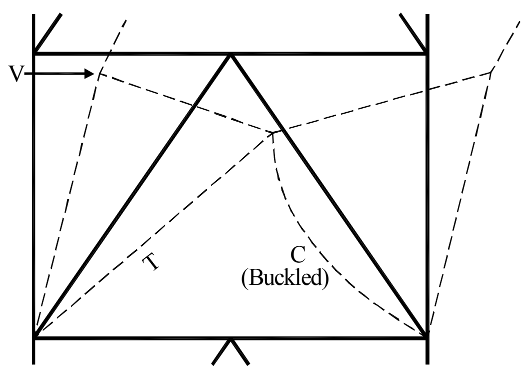
Brace Configurations Illustrations

Note: Recommend concrete tie beams or concrete encased steel beams to tie braced frame base/footings together.

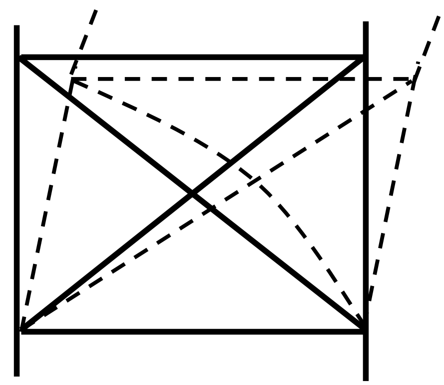
Figure 2-1

When the brace buckles out-of-plane, the single gusset plate is now bending about its weak axis; hinging is occurring in the gusset plate and not the brace. This weak axis gusset plate bending results in significantly reduced residual in-plane stiffness of the brace frame and dissipates less energy than if hinging were occurring in the brace itself.

It should be noted that regardless of the axis of buckling (in-plane or out-of-plane), when the compression brace buckles in a V or inverted V (chevron) braced frame the beam at the mid-span connection must deflect downward (see figure 2-2). This deflection can result in significant damage to the slab system attached to this beam. This type of beam damage is not anticipated in the single-story X brace since the connections are directly to the columns.



Chevron Bracing Postbuckling Stage



X Bracing Postbuckling Stage

Figure 2-2

The preference may be to detail the brace to buckle in-plane, if possible, instead of out-of-plane. This may help minimize non-structural damage to interior stud walls or building perimeter curtain walls, adjacent to or enclosing the brace, which would occur if the brace buckles out-of-plane instead of in-plane. The design engineer should be aware that when a brace buckles out-of-plane, the horizontal displacement out-of-plane at the brace mid-span, perpendicular to the brace, could be significant. Brace buckling deflections ranging from ten to 20 inches can be reasonably expected as the brace length increases from eight to 17 feet, respectively (figure 2-3C). The longer the brace span, the greater the anticipated in-plane or out-of-plane deflection as the brace goes through buckling behavior. This displacement can result in damage to stud wall or other elements which encase or conceal the braces, as previously mentioned. A method for calculating brace deflections is given in the SEAOC *Seismic Design Manual,* Vol. III (Updated for the 2000 IBC). Single-story X braces are expected to have less out-of-plane displacements due to the tension brace helping to restrain the compression brace buckling displacement.

If infill studs occur in-plane above and below the diagonal brace members, the axial stiffness of the stud walls may lead to the brace still buckling out-of-plane. The AISC *Seismic Provisions* contain prescriptive design requirements for out-of-plane buckling of the SCBF brace single gusset plate connection. The direction of brace buckling is the designer's choice.

For a single gusset plate connection with the gusset plates in the plane of the brace, when a brace is designed to buckle out-of-plane, it is imperative that the gusset plate yield line be perpendicular (90 degrees) to the axis of the brace member at each end of the brace (see figure 2-3). There are two reasons for this explained in sections 2.3.1 and 2.3.2 below.

2.3.1 Minimize Brace End Restraints

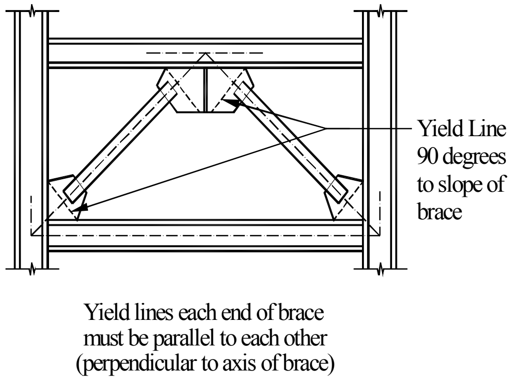
When a brace buckles out-of-plane, it induces out-of-plane bending in the single gusset knife plate connection since the gusset plate has the least stiffness in this direction. The brace cannot buckle (rotate) freely about the gusset plate yield-line hinge unless the gusset plate yield lines at each end of the brace are parallel (see figure 2-3). If the yield lines are not parallel, then more restraint is developed at one end of the brace than the other, resulting in a potential for tearing of the gusset plate with the most restraint or damage to the end of the brace. This could compromise the integrity of the brace-end connection.

2.3.2 Shaping Brace Ends Does Not Change Yield Line Axis

Out-of-plane bending of a gusset plate always occurs about a line perpendicular to the axis of the lever arm (brace in this case), as long as the gusset plate does not buckle under axial load. Shaping the end of the brace member (such as cutting the end of the hollow structural section [HSS] to be at 45 degrees) does not rotate the gusset plate yield line away from being perpendicular to the longitudinal axis of the brace. Instead, shaping the brace end, depending upon location of gusset plate edge restraints, will either move the perpendicular yield line to the tip end of the brace or attempt to warp the straight yield line into a curved shape around the end of the brace. Warping of the gusset plate yield line into a curve shape may not be physically possible, resulting in tearing of the gusset plate to reduce the restraint caused by attempting to bend the plate about a curve.

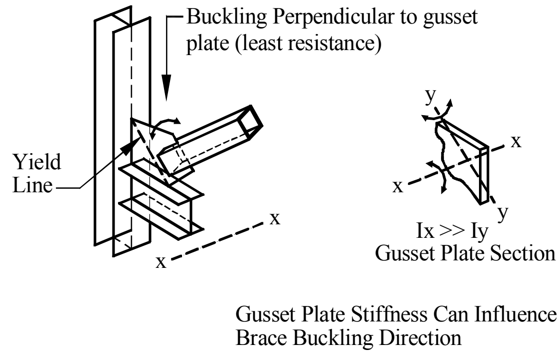
2.3.3 Brace Mid-span Buckling

As the brace begins to buckle (figure 2-3C), hinges develop in the gusset plates and out-of-plane rotation occurs. If the axial force continues to increase, a third hinge will form at the mid-span of the brace.



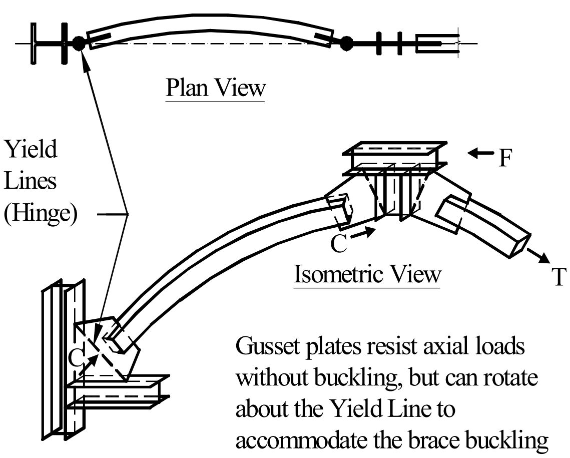
Out-Of-Plane Buckling of Braces

Figure 2-3A



In-Plane vs. Out-Of-Plane
Buckling of Braces

Figure 2-3B



Out-Of-Plane Buckling of Braces

Figure 2-3C

Note that although figures 2-3B and 2-3C illustrate the brace buckling out-of-plane of the brace frame bay, it is also possible to orient the gusset plates so they are 90 degrees to the plane of the brace frame bay (gusset plate sloped, oriented in the flat position and perpendicular to the beam web), thereby causing brace buckling to occur in the plane of the brace frame bay. When the gusset plates are oriented so the brace buckles in-plane, there may still be damage to the partition walls around the brace, especially if the stud wall is constructed as an infill wall with studs framing directly above and below the brace members.

2.4 Columns in SCBF

The UBC and the AISC *Seismic Provisions* both require that columns in SCBF meet the b/t ratios for compression members (per section 2213.7.3 of the UBC or table I-8-1 in the AISC *Seismic Provisions*). The IBC follows the AISC *Seismic Provisions* by direct reference. See table 3-1 in section 3.2 for limiting b/t ratios for various column section shapes.

Design loads for columns in the UBC are specified in section 2213.5.1, which is referenced from section 2213.9.5. UBC section 2213.5.1 refers to the LRFD load factor equations in UBC section 1612.2 and in addition, in UBC seismic zones 3 and 4, the columns must have the strength to resist axial loads from two additional load combinations. The IBC refers to the AISC *Seismic Provisions* directly, and the AISC *Seismic Provisions* refers to the AISC LRFD specification. Section A4 of the LRFD specification refers to ASCE-7 for load combinations.

2.4.1 Column Splices

Both the UBC and AISC Provisions (and the IBC by reference to the AISC Provisions) require that SCBF column splices develop the full shear strength and 50 percent of the full moment strength of the smaller of the columns at the column splice. Splices shall be located in the middle one-third of the clear column height. In addition, in UBC seismic zones 3 and 4, the UBC requires that column splices have sufficient strength to develop the column forces determined from the additional load combinations in section 2213.5.1.

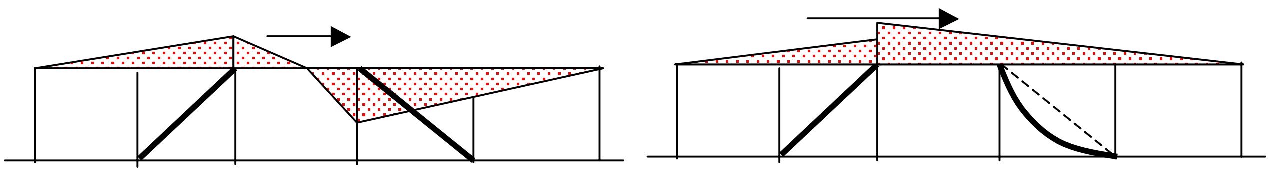
Welded column splices subject to net tensile stress using the applicable building code load combinations with an amplified seismic load must be made using weld filler metal with Charpy V-Notch toughness. See the AISC provisions.

If partial-joint-penetration welds are used, they shall be designed to have 200 percent of the required strength.

If full-penetration welds are used, beveled transitions are required to reduce stress rising “notches” and eliminate reentrant corners where cracks can develop. See the AISC provisions 8.4a and the commentary to 8.4a.

2.5 Drag Connections

Drag members should be provided over the entire length and width of the building diaphragm to transfer forces to the braced frame bays. Drag members within the dragline must be capable of resisting both compression and tension forces and have adequate connections to the braced frame bays themselves. Note that draglines should be checked for seismic forces in both directions (tension and compression), assuming that the compression braces in the braced bays have buckled making the effective dragline length longer by the distance between the braced bays in some cases (example: single diagonal braced bays with buckled compression braces).



Drag Line Diaphragm

Drag Line Diaphragm with Buckled brace

Figure 2-4

Compression buckling of the brace causes a redistribution of brace forces (within the braced bays) to the tension braces, which temporarily take the entire lateral load until the direction of the earthquake lateral force reverses. This redistribution of brace forces results in a redistribution of axial forces in the drag members, and the drag members should be checked for these conditions.

The steel beams in the dragline should be designed as non-composite for gravity loads, and probably should not be smaller than a W16x member to allow for enough bolts in the connection. The longer the draglines, the larger the beam size will typically be. Headed shear studs (Nelson studs) are typically used to transfer the seismic inertial forces of the diaphragm into the beams along the dragline.

Because of Ω_o, the axial loads are very large in the dragline beams, and may require the beam-to-beam or beam-to-column connections along the dragline to have complete penetration welds of the beam flanges and web or lap plates to transfer the axial loads across the connection. The column continuity plates in this case will also require complete penetration welds to the column flanges. The continuity plate should be detailed to prevent welding from occurring in the "k" area of the column and the designer may want to review the *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* reference (FEMA-350) for continuity plate details. Multiple rows of bolts in the beam/column connection are possible in the beams, but block shear of the beam web could be inadequate compared to the design forces.

The diaphragm typically stabilizes the top flange of the drag beam. The bottom flange of the drag beam will typically require periodic bracing along its length to prevent localized buckling of the beam flange (r_y axis) when the drag force axial load causes compression and gravity loads cause bending.

3.0 LIMITATIONS OF BRACING SYSTEMS

3.1 Building Height

The AISC *Seismic Provisions* defers to the applicable building code for building height restrictions. The Uniform Building Code places height restrictions for the various lateral resisting systems used in building design in seismic zones 3 and 4, unless a Dual System is used, which has no restrictions on building height. The UBC limits a building using braced frame systems in either of the two orthogonal directions to the following heights:

OCBF:	160 feet
SCBF:	240 feet

The IBC 2000 has no height restrictions for buildings in seismic categories A, B, and C for both OCBF and SCBF. Height restrictions differ for seismic categories D, E and F as follows:

	Seismic Category		
	D	E	F
OCBF:	160 feet	100 feet	100 feet
SCBF:	160 feet	160 feet	100 feet

The height restrictions have been further restricted in the IBC 2003 table 1617.6:

	Seismic Category		
	D	E	F
OCBF:	35 feet	35 feet	Not permitted
SCBF:	160 feet	160 feet	100 feet

There are footnotes to table 1617.6 that allow for some exceptions. The reduction in height limits for the OCBF is due to the lack of brace buckling ductility in the connections and lack of prescriptive requirements for the brace frame members (example: b/t ratios, Kl/r ratios, load combinations, etc.)

Some jurisdictions in California, such as the City of Los Angeles Department of Building and Safety, have already adopted the more stringent height limitation of 35 feet for OCBF (2002 City of Los Angeles Building Code).

3.2 Bracing Compression Member Restrictions

3.2.1 Width-Thickness Ratios

Since braced frame members are primarily axially loaded, the UBC has width-thickness ratio restrictions on compression elements used in braces requiring them to comply with compact shape requirements of division II (section 2206) and III (section 2208) which respectively refer the reader to the AISC LRFD and ASD specifications, table B5.1. The 2002 AISC *Seismic Provisions* refers to table I-8-1 for width-thickness ratios "λ_{ps}" for compression members. Listed in table 3-1 below are the limiting restrictions for typical bracing elements, which are the same for both OCBF and SCBF in the 1997 UBC.

Shapes	**Width-Thickness Ratios (1997 UBC)**
Wide Flange	$b/2t = 65/\sqrt{F_y}$ Beams $= 52/\sqrt{F_y}$ Columns (SCBF)
	$d/tw = 253/\sqrt{F_y}$
Angles	$b/t = 52/\sqrt{F_y}$
Pipes, Round HSS	$D/t = 1300/\sqrt{F_y}$
Rectangular HSS (Tubes)	$b/t = 110/\sqrt{F_y}$

Shapes	**Width-Thickness Ratios (2002 AISC Seismic Provisions)**
Wide Flange	$b/2t = 0.30(\sqrt{E_s/F_y} \approx 51/\sqrt{F_y}$
	For $(P_u/\phi_b P_y) \leq 0.125$ $h/tw: = 3.14(\sqrt{E_s/F_y})(1 - 1.54(P_u/\phi_b P_y))$ $(\text{range} = (3.14 \text{ to } 2.53)\sqrt{E_s/F_y}$ $\approx (535 \text{ to } 431)/\sqrt{F_y}$ For $(P_u/\phi_b P_y) > 0.125$ $h/tw: = 1.12(\sqrt{E_s/F_y})(2.33-(P_u/\phi_b P_y))$ $(\text{range} = (2.47 \text{ to } 1.49)\sqrt{E_s/F_y}$ $\approx (421 \text{ to } 254)/\sqrt{F_y}$
Angles	$b/t = 0.30\sqrt{E_s/F_y} \approx 51/\sqrt{F_y}$
Pipes, Round HSS	$D/t = 0.044\sqrt{E_s/F_y} \approx 1276/\sqrt{F_y}$
Rectangular HSS (Tubes)	$b/t = 0.64\sqrt{E_s/F_y} \approx 109/\sqrt{F_y}$

Table 3-1 Brace Member Width-Thickness Requirements

The use of the b/2t ratio from the 2002 AISC *Seismic Provisions* is strongly recommended for wide flange shapes instead of the 1997 UBC b/2t ratio, which is less restrictive for beams. The h/tw limit for wide flanges in the 2002 AISC *Seismic Provisions* will be closer to 1.49 $\sqrt{E_s / F_y}$ since the actual expected brace axial force will be approaching its buckling strength. The 2002 AISC *Seismic Provisions* have removed the more restrictive b/t ratio restrictions on OCBF compression members. In the authors' opinion, the designer should still consider using the SCBF more restrictive b/t ratios for OCBF. The State of California Building Code (CBC) still requires the more stringent b/t ratios for OCBF since it is based upon the 1997 UBC.

Some engineers fill the HSS and pipe sections with concrete to prevent local buckling of the brace wall. This results in a composite brace member with additional axial compression capacity. The brace frame design using composite brace members is beyond the scope of this *Steel TIPS*. The designer can review the AISC 2002 *Seismic Provisions (*part II) for information regarding composite brace frame design.

3.2.2 Unbraced Length of Brace Member

The unbraced length of bracing members (Kl/r) is also restricted:

$$\text{OCBF} = 720/\sqrt{F_y}$$

$$\text{SCBF} = 1000/\sqrt{F_y}$$

The compact section and unbraced length limitations effectively restrict when certain types of bracing members can be used, as listed below. The 2002 AISC *Seismic Provisions* have revised the bracing member Kl/r ratio limit equations to include the Steel Modulus of Elasticity (E_s) but still have the same previous limits. The Kl/r restriction on OCBF's braces have been relaxed to the LRFD limit of Kl/r limit of 200. V (chevron) braces in OCBF have a Kl/r limit of $4.23\sqrt{E_s / F_y} \approx 720/\sqrt{F_y}$. The SCBF have a Kl/r limit of $5.87\sqrt{E_s / F_y} \approx 1000/\sqrt{F_y}$.

The SEAOC *Seismic Design Manual*, Vol. III, (updated for the 2000 IBC) has additional information regarding the appropriateness of a given K value for Kl/r values to use in brace design considering the end restraints of the brace (fixed or pinned).

3.2.3 Maximum Brace Sizes and Lengths

The maximum allowable brace member sizes and lengths used in braced frames are limited by both the b/t ratios and the KL/r ratios restrictions for the various available steel shapes.

Angle Braces:

L8x8x1-1/8 (b/t = 7.11 < 7.35) F_y = 50 ksi
L8x8x1 (b/t = 8.0 < 8.67) F_y = 36 ksi
L6x6x1 (b/t = 6.0 < 7.35) F_y = 50 ksi
L6x6x3/4 (b/t = 8.0 < 8.67) F_y = 36 ksi

Length (from l/r limit):

2-L8x8x1 (F_y=36): $l = (720/\sqrt{F_y})(r_x)$ = 24.4 feet (OCBF)

2-L8x8x1 (F_y=50): $l = (1000/\sqrt{F_y})(r_x)$= 33.6 feet (SCBF)

The steel yield strength (F_y) affects the allowable angle sizes, as can be seen above. As the steel strength increases from 36 ksi to 50 ksi, the thickness of the angle leg must increase.

Tube Braces (HSS Sections):

The definition of b/t is different for the 1997 UBC and the 2002 AISC *Seismic Provisions*, so maximum brace sizes are code dependent. The UBC defines "b" as the out-to-out dimension of the tube whereas AISC defines "b" as the flat width (distance between the radii of the tube corners). AISC publishes the b/t values for the tube sections in the *Hollow Structural Sections Connections Manual*, Chicago, 1997, and the LRFD Manual, 3rd Ed. The engineer needs to check with local building officials as to which criteria will be acceptable for design.

2002 AISC *Seismic Provisions* (F_y = 46 ksi):

Rectangular HSS Sections (Tubes):

HSS 10x10x5/8 (b/t = 14.2 < 16.2)
Length (from l/r limit)= 33.4 feet (OCBF)
= 46.4 feet (SCBF)

HSS 8x8x5/8 (b/t = 10.8 < 16.2)
Length (from l/r limit) = 26.4 feet (OCBF)
= 36.7 feet (SCBF)

HSS 8x8x1/2 (b/t = 14.2 < 16.2)
Length (from l/r limit) = 26.9 feet (OCBF)
= 37.3 feet (SCBF)

1997 UBC (F_y = 46 ksi and using nominal wall thickness and AISC ASD Manual, 9th Ed. "r" values)

HSS 10x10x5/8 (b/t = 16.0 < 16.2)
Length (from l/r limit) = 33.4 feet (OCBF)
= 46.4 feet (SCBF)

HSS 8x8x1/2 (b/t = 16.0 < 16.2)
Length (from l/r limit) = 26.8 feet (OCBF)
= 37.2 feet (SCBF)

Currently, the typical wall thickness of commercially produced larger tube sections (12x12, etc.) do not exceed 5/8 inch; therefore, they cannot be used unless the tube walls are reinforced with additional flat plates.

The current wall thickness (t) of rectangular HSS (tube) sections produced by the steel mills is approximately 93 percent of the published values in the AISC ASD Manual, 9th Ed., and LRFD Manual, 2nd Ed., and is therefore no longer correct. The correct wall thickness values are published in the AISC *Hollow Structural Sections Connections Manual* and in the LRFD Manual, 3rd Ed. When calculating the actual tensile ($P_n=R_yA_gF_y$) and compressive ($P_n=A_gF_{cr}$) capacities for the brace, the actual area should be used (≈93 percent of AISC ASD Manual, 9th Ed.). Using the nominal tube sizes in the AISC ASD Manual, 9th Ed. will result in conservatively higher connection tensile forces and an overestimation of the brace compression capacity. The reduced area also affects the "r" values used in checking slenderness (KL/r) for the tube sections, but this change is very minor.

<u>Pipe and Round HSS Braces</u>:

12-inch standard (D/t = 34 < 37.1 for F_y = 35 ksi)
Length = 43.8 feet (OCBF)
= 60.8 feet (SCBF)

12.75 x 0.5 HSS (D/t = 27.4 < 30 for F_y = 42 ksi)

Currently, the typical wall thickness of commercially produced larger round HSS pipe sections does not exceed 5/8 inch; therefore, they cannot be used unless reinforced.

3.2.4 Minimum Brace Sizes

The designer is also reminded that the use of tubes with thinner wall sections is limited by the b/t ratio in the 1997 UBC and California Building Code.

Example: 1997 UBC: HSS 8x8x3/8 b/t = 21.333 > 16.2 (No Good)
HSS 6x6x1/4 b/t = 24.00 > 16.2 (No Good)

The 2002 AISC *Seismic Provisions* have removed the b/t restriction for OCBF, therefore framing members with a thinner wall thickness (example: HSS 6x6x1/4) could be used where the 2002 AISC provisions are allowed.

3.2.5 Story Height Recommendations

With the restrictions on bracing member sizes, using bracing members other than wide flange sections will only be practical for shorter buildings where the associated base shears are lower, or for the upper floors of taller buildings. Because of simplicity and detailing economics, once a type of bracing member shape has been selected, it can be more economical to continue using the same type of member for two or three floor levels instead of changing bracing member types and associated details at each floor level.

The same size brace should not be used at every floor level of a taller multistory building. The braces at the upper floor levels will be oversized, resulting in more of the seismic forces being concentrated at the first floor level of the building where the brace (same size brace as floors above) will have the least reserve capacity. This does not lead to an efficient or economical building design because of the number of oversized braces. If a capacity design approach is used for the brace frame members and connections, this can lead to substantially increased foundation sizes and column uplift forces.

Table 3-2 gives approximate guidelines for the suitability of specific types of bracing members based upon building height.

Member Shape	**Number of Stories**
Wide flange bracing members	Unlimited
Rectangular HSS (tubes)	Approximately 8 stories
Round HSS and pipes	Approximately 8 stories
Double angles, quad angles	Approximately 7 stories for double angles; possibly taller for quad angles (star angles)

Table 3-2 Guidelines for Types of Bracing Members based on Building Height

The number of stories for a particular brace type depends upon the number and lengths of braces used in the building; therefore, the actual number of stories may be more or less than what is shown in the table.

3.3 Column Restrictions

No special restrictions are placed on the column flange $b_f/2t_f$ for OCBF. SCBF though, to ensure plastic performance, require the $b_f/2t_f$ of wide flange column shapes to be less than $52/\sqrt{F_y}$ (1997 UBC) and $0.30\sqrt{E_s/F_y}$ (2002 AISC *Seismic Provisions*) as shown in table 3-1 above. The 2002 AISC *Seismic Provisions* also limit the SCBF Wide Flange column section h/t_w ratio, which will be closer to

$1.49\sqrt{E_s/F_y}$ since the expected axial force in the column will be approaching its buckling strength.

3.4 Beam Restrictions

The 1997 UBC places no specific framing requirements upon brace frame beams unless they are used in V or inverted V (chevron) brace frames. Though not required by the 1997 UBC, the authors strongly recommended that V brace frame beams also satisfy the same width/thickness requirements (compact section requirements) as the braces to help prevent localized buckling. This would be especially true if a portion of the beam length is considered to be unbraced by a diaphragm, such as may occur with a flying beam going through a floor opening.

The 2002 AISC *Seismic Provisions* require SCBF beams to comply with the same restrictions as SCBF braces for b/t and h/t_w limits (Footnote d to AISC 2002 *Seismic Provisions* table I-8-1), which was not previously required in the 1997 AISC *Seismic Provisions* or its supplements. The 2002 AISC *Seismic Provisions* also have the same beam design requirements as the 1997 UBC when using Vee or Inverted vee brace frames.

4.0 PROS AND CONS OF VARIOUS BRACING SYSTEM CONFIGURATIONS

4.1 General Comments

The selection of a specific brace frame configuration is often dependent upon the location. Architectural restrictions may prevent the use of a V or X bracing, limiting the engineer to the use of single diagonal braces. In most cases, the architect will prefer that you use the V braces since they allow for a door to be placed in the braced frame bay. V or inverted-V brace systems should perform adequately if the beam is properly designed for the brace unbalanced vertical component forces as required for the SCBF. The authors' preference is to still recommend the use of the single-story X, the two-story X or single diagonal braces in both SCBF and OCBF over the single-story V brace because they are expected to have better performance (see the SEAOC *Recommended Lateral Force Requirements and Commentary*, 7th Ed., 1999 [also known as the Blue Book]). OCBF V or inverted-V brace systems are not expected to perform well since the beam is not required to be designed for the unbalanced vertical component forces resulting from brace buckling and tension yielding.

4.2 Single Diagonal Bracing

The efficiency of the braced frame, because of code redundancy requirements, is typically dependant upon the number of diagonal members provided. For an equal number of braced bays, the single diagonal will be the least effective since it has half the number of diagonal members per bay. When the bay width is less than 15 feet, a single diagonal brace or single story X bracing should be considered. (See figure 2-1A).

In the authors' opinion, two bays of single diagonal braced frames (braces in opposite directions) are preferable to a single bay of V or inverted V braces. The columns should also be checked for compression buckling due to the vertical component of the brace at tensile yield strength level ($A_gR_yF_y$). When checking the column compression capacity, it is left to the engineer to determine how many floor levels of buckled braces above the specific floor level being evaluated to include in the design (the number of tension brace vertical components to consider).

The most conservative approach would be to consider all tension braces framing to the same column as having yielded at the floor levels above the floor level being checked. The authors recommend that the tension braces at all floor levels attached to the same column be considered as yielded for buildings three stories or less in height when designing the column. This column seismic axial force is typically greater than the seismic axial force (Ωo)E required in the special load combinations for SCBF column design, regardless of building height. A non-linear analysis should probably be used for four-story and taller buildings to determine the extent of tension yielding braces and the impact on columns from overturning forces if all tension braces attached to the column are not considered to be yielded. Gravity loads are additive to the seismic loads.

4.3 Single Story X Braces

The single story X brace, though having two diagonal members, usually is not practical because of doorways and other openings which conflict with the X brace. There is also the additional detailing for the splicing of the diagonal members where they cross each other. The splicing detail is typically easiest where angle bracing can be used (see figure 2-1B) since it has a single gusset plate. A single knife gusset plate can be used for HSS and pipe sections as well, but for HSS rectangular sections it may be easiest to use lap plates each side of the brace splice.

Because of the more truss-like behavior of the single-story X brace, two-story X brace and single diagonal brace compared with single-story or stacked V or chevron braces, the designer may want to consider checking the columns for axial compression to be able to resist the vertical component of the brace tensile capacity ($A_gR_yF_y$) plus gravity loads. This is not a building code requirement, and the brace bay columns are supposed to be checked with special load combinations that include Ω_o, but the resulting column design force is usually less than the vertical component of the tensile yield capacity of the brace. When the column is designed for this higher force level, then the column will not be the weakest link in the brace frame bay that could lead to a collapse.

The steel brace frame example in the SEAOC *Seismic Design Manual,* Vol. III for the 2000 IBC provides information for single-story X bracing. This design example includes appropriate K factors and end fixity (pin, fixed) factors based upon connection detailing to consider in the design of both the X brace splice connection and the brace members themselves.

4.4 V and Inverted V (Chevron) Braces

The V or inverted V (chevron) brace configurations (referred to as V braces hereinafter) are typically the most efficient architecturally since they provide two braces per bay and are flexible to allow for openings at the mid-span of the braced bay. Multistory buildings can also utilize the V brace in a two-story X brace fashion (V braces inverted in every other story) as illustrated in the bracing configuration details (see figure 2-1D).

The disadvantage of V bracing systems is the requirement to design the beams for the unbalanced loading that occurs when the compression brace buckles and the tension brace pulls down on the beam. This potential failure mode results in much larger beams than would be required in other brace configurations. This is the primary reason to use multistory X bracing (single-story X or two-story X) instead of single-story V bracing – so the beam does not have to be designed for the unbalanced loading resulting from compression brace buckling.

If V braces are used, the ends of the SCBF beams are generally assumed to be “pinned-pinned.” In the authors’ opinion, this is a very conservative assumption which leads to large beam sizes since most of the beam bending moment is due to the Q_b (unbalanced) load. In reality, if the gusset plates at the ends of the beam can be shown by the engineer to provide partial or full fixity to the ends of the beams, a smaller moment can be calculated by taking advantage of the end fixity. Instead of calculating the effect of Q_b with $M = Q_bL/4$ (pinned ends), the moment “M” could be calculated as $M = Q_bL/6$ to $Q_bL/8$ (with partially to fully fixed ends), substantially reducing the portion of the beam moment due to Q_b. If the engineer could show that the beam-end moment can be transferred to the gusset plate (along its length at the interface with the beam flange) and the beam-end connection, then beam-end fixity could be assumed.

For example, consider the second floor beam of a two-story stacked V brace frame. Fixity might be shown by resolving the beam-end moment into a couple T = C = (M/beam depth) and determine the shear that would be developed due to “T” (along the gusset plate/beam top flange interface) and the compression force “C” that would be developed in the beam bottom flange if welded to the column flange. The T force in the gusset plate must be transferred to the column via the gusset plate-column connection. In this case, the effective length of the beam for calculating the beam moments could be reduced by half the beam depth at each end, further reducing the beam bending moments. There is a need for more research regarding this issue.

The minimum bay width for V bracing should be about 20 feet for the normal floor height of 11 to 13 feet. The slope of the braces should be between 40 and 50 degrees. Brace slopes greater than 50 degrees make it difficult to swing the brace into position if the beams have already been placed. Most of this is a function of how long the brace is relative to the gusset plate connections. For long braces, and relatively short gusset plates, if the floor beam is placed after the V bracing is placed, then smaller bay widths can be used.

Where both braces frame to the underside of the beam, the use of a single gusset plate is recommended instead of two individual plates, due to potential problems with alignment and heavier welds when using two plates.

4.5 Two-Story X Braces

The behavior of the two-story X brace requires special consideration for gravity loads and seismic forces (see figure 2-1F). Special attention should be given to the columns, as mentioned in the single-story X brace section, to be certain they are able to resist the vertical component of the brace tensile yield force without buckling.

4.5.1 Gravity Loads

The two-story X brace (figure 2-1F) develops significant dead and live load axial forces in the braces due to restraint resulting from the brace/column connection. The vertical component of the brace is attempting to prevent column shortening due to the gravity loads. These forces become very significant, since now dead and live loads are being accumulated at each floor level as you go down the height of the building. The dead and live load horizontal component of the brace force must ultimately be transferred to the foundation and may even exceed the lateral forces due to wind or seismic activity in taller buildings in areas of lower seismicity.

As a comparison, consider the bracing dead and live loads in a multistory building, where the single-story V bracing (or inverted V bracing, see figures 2-1D and 2-1E) is repeated at each floor level. In the single-story V, bracing gravity forces originate from the beam mid-span connection and are transferred to the beam/column connection at the floor below. The vertical force component is transferred into the column and the horizontal force component is transferred into the beam. Since the two braces of the V are in opposite directions, the beam now acts as a tension tie between the two columns. Therefore, the single-story V braces are only supporting gravity loads from a single floor level.

If the columns at each end of the single-story V brace are of different sizes, you will see some accumulation of gravity loads in the braces from each floor level. This is primarily due to differences in column axial shortening between the two columns.

4.5.2 Lateral Forces

When evaluating the two-story X brace (figure 2-1F), the lateral forces in the X brace frame beams will typically be very small. At the floor levels where the braces intersect at the beam mid-span, the beam will be acting as a drag strut to the brace connection. At the floor levels where the two-story X brace intersects at the beam/column connection there will typically be very small axial forces in the X brace frame beam, except for possible drag forces.

The small axial forces are due to the fact that the lateral drag forces go directly into the brace and generally do not need to be dragged through the X brace frame beam to the other column. The exception would occur when there are floor openings along the length of the beam or two-story X braces occur in adjacent bays requiring forces to be dragged to the interior column brace connection. Once the brace frame begins to go inelastic though, there is a redistribution of forces and the axial loads in these brace frame beams can become very large.

4.6 Zipper Columns

The zipper column is another approach for V brace systems whereby the beam of the SCBF can be designed for a lesser force than the unbalanced force that occurs when a V brace buckles. The theory behind the zipper column is that when a brace buckles, the zipper column will carry the vertical component of the unbalanced force to the beams of the floors above. The zipper column is intentionally omitted from the first floor level so the unbalanced buckled brace force is redistributed to the upper floor levels through the zipper column (see figure 2-1H).

The engineer, when using the zipper column approach, should give consideration to more than a single brace buckling in any particular brace frame, especially in taller buildings. The actual practicality of using the zipper column may be limited due to architectural constraints. There is also concern about supporting the zipper column when one of the first floor V braces buckle. Design guidelines are being developed for the AISC *Seismic Provisions* for Zipper columns.

4.7 K Braces

K braces are not permitted for SCBF because the columns could be subjected to unbalanced forces from the braces that could contribute to column failures. K braces are not allowed for OCBF in the 1997 UBC unless using the One- and Two-Story exception (section 2213.8.5). K braces are not allowed for OCBF in the 1997 AISC *Seismic Provisions* (and in the IBC by reference to the AISC Provisions) unless using AISC *Seismic Provisions* section 14.5, Low Buildings (roof structures and two story and less buildings). The 1997 AISC *Seismic Provisions* (Supplement #2) and the 2002 AISC *Seismic Provisions* both allow the use of K bracing for OCBF. The story height limitations for OCBF are no longer provided in the AISC *Seismic Provisions*, but instead are in the adopted Model Building Codes (example: IBC, NFPA 5000). It is the authors' opinion that K braces should never be used in areas of moderate to high seismicity for OCBF (see figure 2-1C), and that if used, the K braced buildings be limited to roof structures and buildings no more than two stories in height, including areas of low seismicity.

4.8 A Word About Buckling Restrained Braced Frames (BRBF)

There is an alternative bracing system to SCBF that has some advantages over SCBF. This system utilizes buckling restrained braces commonly called unbonded braces. Essentially, the concept of unbonded braces is the prevention of buckling of a central core steel brace encased in a steel tube filled with concrete or grout with a slip interface between the core brace and the concrete or grout in the encasing tube. These braces have equal stiffness and strength in both the tension and compression modes without deterioration of strength or stiffness in the braces over many cycles of tension/compression.

BRBF braces, because of their equal performance in either the tension or compression mode, have an advantage when used in a V configuration because there is no unbalanced vertical component from compression brace buckling as could occur in a SCBF. This allows the beam at the apex of the braces to be smaller and to not have to be designed for the unbalanced vertical component, or at least to be designed for a much smaller vertical component load. Also the cyclic hysteric curves for seismic energy dissipation are full (not pinched) without the deterioration seen in SCBF after several compression/tension cycles where the compression braces repeatedly buckle and straighten. Unbonded braces have been extensively tested and are now being used in new and retrofitted buildings.

BRBF braces are proprietary and can cost more than SCBF braces, but as this *Steel TIPS* is written, there are now several manufacturers of BRBF braces so prices should become more competitive with SCBF. There will be a *Steel TIPS* on BRBF published in 2004.

5.0 SELECTION OF BRACING MEMBERS

5.1 Pros and Cons of Various Shapes of Bracing Members

Eligible bracing members sections are determined from the stiffness and strength requirements of the lateral system. Typical members used for bracing include structural tubes, wide flange sections, single angles and multiple angles.

Many factors, as discussed in table 5-1 below, will affect the ultimate selection of a bracing member from the available members that satisfy the design requirements.

Strong consideration should also be placed on the size of the project, the capabilities of the fabricator and the likely type of field erection techniques. For instance, on a large project, a fabricator may favor wide flange braces to take advantage of a steel mill order for the identical wide flange section used throughout the remaining structure. Since that shop also wants to take advantage of their automated equipment, the field-bolted connection may be preferred.

Type of Bracing Member	**Advantages**	**Disadvantages**	**Other**
Structural Tubes (Rectangular HSS Members)	• Aesthetic as an architecturally exposed element. • Higher yield strength than angles. • Very efficient for keeping a brace confined within the column web space.	• The b/t ratio requirement limits the number of sections available and also heavily favors the square members. • Generally requires a welded connection. • Low cyclic fatigue life as compared	• Has a higher cost per pound of bulk material, but these costs are often offset when considering the total installed cost. • May require net section reinforcement plates.

Type of Bracing Member	**Advantages**	**Disadvantages**	**Other**
		to wide flange sections due to cold forming stress concentrations along HSS side wall radius bends.	
Pipes and Round HSS Sections	• Aesthetic as an architecturally exposed element.	• Difficult to align slots at each end of brace so they are not rotated relative to each other • Generally requires a welded connection.	• Most HSS round sections typically can be used. • Traditional pipe (standard, x-strong, xx-strong) satisfy the circular section criteria. • May require net section reinforcement plates.
Wide Flange	• Many sections commonly available. • Could be considered aesthetic as an architecturally exposed element. • Higher strength than angles. • Has the highest capacity (due to large selection).	• The connections to fully engage the cross section can become complicated. • Generally favors a field-bolted connection.	• Bolted connections will require net section reinforcement plates.
Single Angle	• Satisfies small loads. • Easy to install. • Commonly available.	• Limited member selection. • Has a relatively poor radius of gyration (r).	• Bolted and welded connections will likely require reinforcement plates to meet net section and shear lag requirements.
Multiple Angle - Double Angle	• Satisfies up to medium load demands.	• Limited member selection. • Has a relatively	• Bolted and welded connections will likely require

Type of Bracing Member	Advantages	Disadvantages	Other
or Quad Angle (Star Angle)	• Commonly available. • For angle configurations spaced ¾ inches apart, axial loads charts are available in the Steel Manual. • As the design permits, Quad-Angles can be "peeled" back to Dbl-Angles to preserve connection details and minimize the number of bracing members used.	poor radius of gyration (r). • Although the individual angle is easy to install, a multiple angle has many pieces to handle increasing the installation cost of each braced bay.	reinforcement plates to meet net section and shear lag requirements.

Table 5-1 Pros and Cons of Various Brace Shapes

On a small project, there may be little preference for the bracing member itself, but field-welded connections are used to ease field fit-up and minimize the detail required for the fabrication shop drawings. This would tend to favor structural tubes and angles as braces.

On other projects where working quarters are tight or the use of cranes is limited, single, double or quad angles may be the most feasible. Even with the increased number of members to fabricate, handle and erect, the individual smaller pieces can be more easily manipulated into place.

5.2 Low Cycle Fatigue

The steel mill manufacturing processes to produce the steel sections and the steel shop fabrication methods to prepare the steel members can influence the low cycle fatigue behavior of the particular brace member. During an earthquake, the building's lateral force resisting system will undergo many load reversals (cycles) of varying stress levels due to the applied seismic inertial forces for the duration of the earthquake. As the earthquake subsides, the magnitude of the building horizontal displacements will decrease to zero as the building eventually cycles back to rest.

The designer is encouraged to determine if there is a need to establish the importance of the fatigue life of the brace members for their particular project. The current building codes do not require fatigue analysis of any of the lateral resisting systems since they must only meet life-safety requirements for what is generally considered to be a one-time event, such as a building code design level or larger earthquake. As part of performance-based design though, fatigue may be an issue to consider, especially if the building is to be operational immediately after an earthquake.

Rectangular HSS shapes have shown poor performance during cyclic axial loading tests that extend into the steel yielding and inelastic range. The poor performance is associated with the original cold working of the steel to form the hollow rectangular shape, which becomes more brittle at the sidewall corner radius bends, the location where fractures typically first appear. Round HSS sections and pipes are expected to perform better since they undergo less cold working to achieve their round shape, but in the authors' opinion, more research should be done on round shapes. The rectangular HSS brace sections for SCBF have more stringent b/t ratios to help improve their performance in the inelastic range during an earthquake.

The wide flange and angle sections are expected to have better fatigue capabilities than rectangular HSS sections since they undergo less initial cold working in the production of the wide flange shape. The wide flange shape is, in the authors' opinion, a more reliable shape for a brace member than a rectangular HSS shape relative to low cycle fatigue and fracture.

6.0 BRACING CONNECTION DESIGN AND DETAILING CONSIDERATIONS

6.1 Connection Design Considerations

In the interest of keeping this *Steel TIPS* from getting too long, most of the discussion is focused on single gusset plates in the plane of the braces to connect pipe or tube braces to beams and columns. Connections for other brace sections such as wide flanges, angles, and channels are not discussed in detail in this *Steel TIPS*, but the general approach presented can be applied to these other connection types. See figure 6-1 for examples of a few generic connection types for these other brace sections. There are many other possibilities that are not shown in this figure.

The designer should realize that there are a large number of other possible connections and should refer to applicable codes and design references when designing and detailing these other connection types. There may be other aspects of the other connection types that will need to be checked.

A very important point to remember is that if the braces are expected to have enough compression force in them to buckle, the braces and their end connections must be designed and detailed so that three hinges can form: one at the center of the brace and one at each end. The end connections must be detailed so that bending can occur either in the gusset plate or in the brace itself. If hinging is to occur in the braces, the end connection must be strong enough to force repetitious ductile hinging to occur in the brace.

Figures 6-1A, B, C, and E show examples of end connections detailed for out-of-plane hinging to occur in the gusset plate. Figures 6-1D and F are detailed so hinges will form in the brace itself. Also, figure 6-1A shows the typical work-point location where the column, brace and beam centerlines intersect. An optional work-point location at the corner of the gusset point is also shown. This work-point location would result in a smaller gusset plate than shown, but the eccentricity created by the centerlines not meeting at a single point would result in a moment that would need to be taken by the column and/or beam. The gusset/beam and gusset/column interface forces and moments for this case would be determined using the Uniform Force Method, special case 2 (see section 6.2.5).

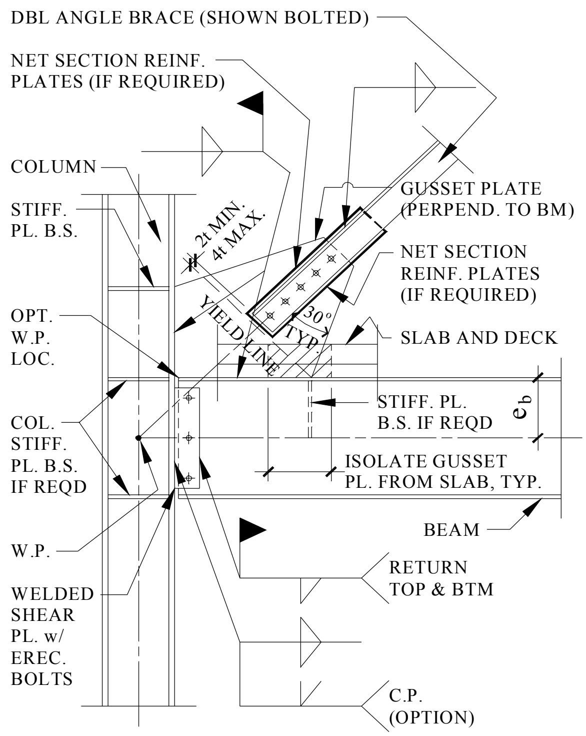


Figure 6-1a

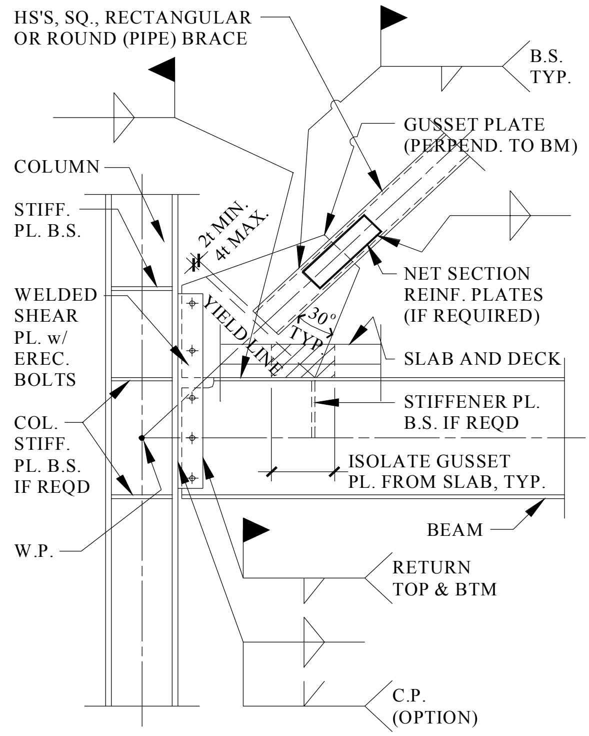


Figure 6-1b

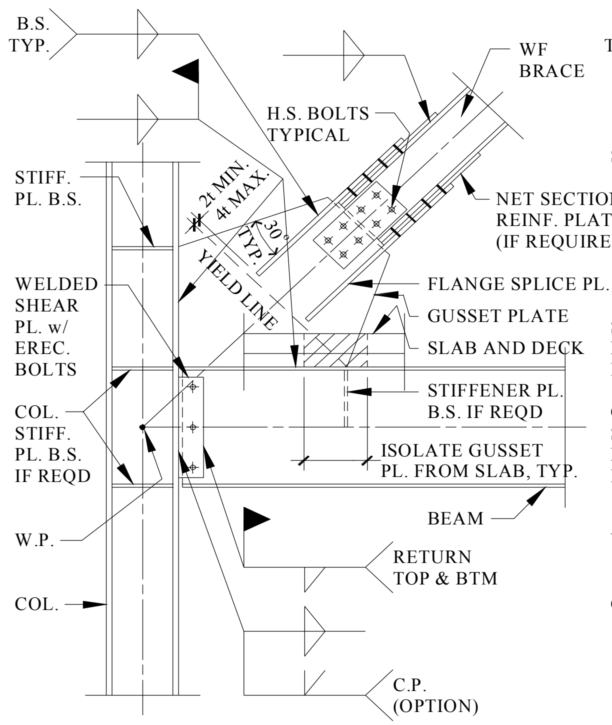


Figure 6-1c

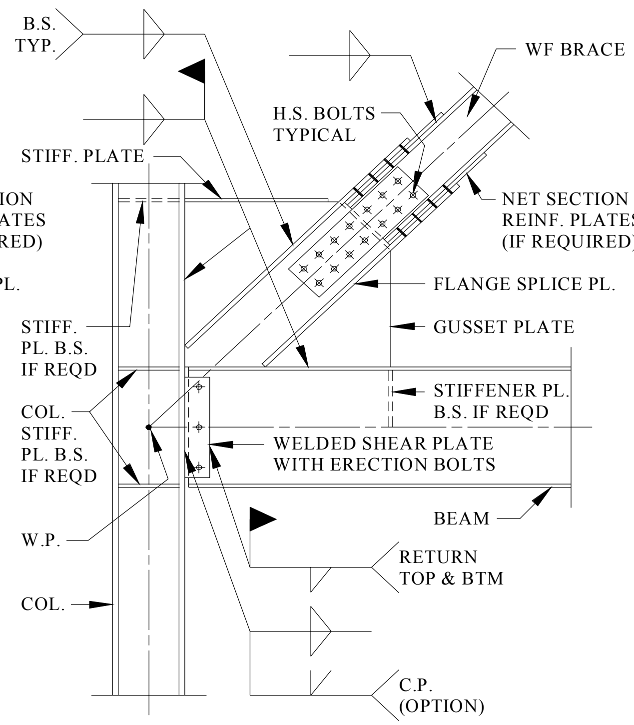


Figure 6-1d

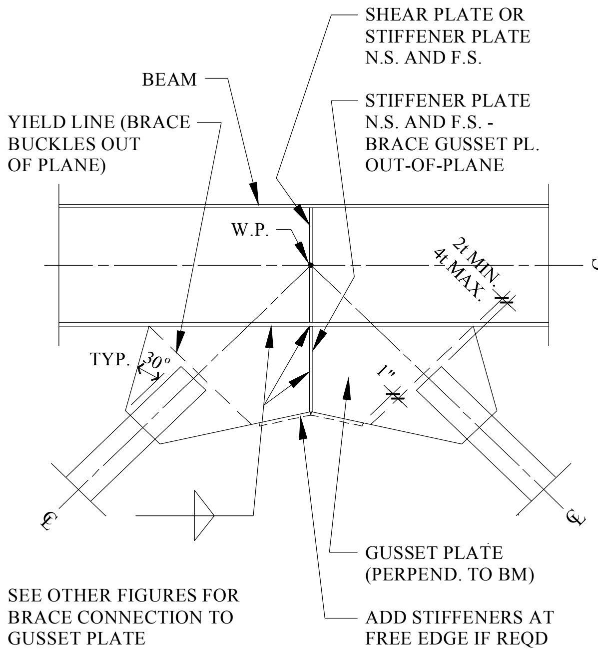


Figure 6-1e

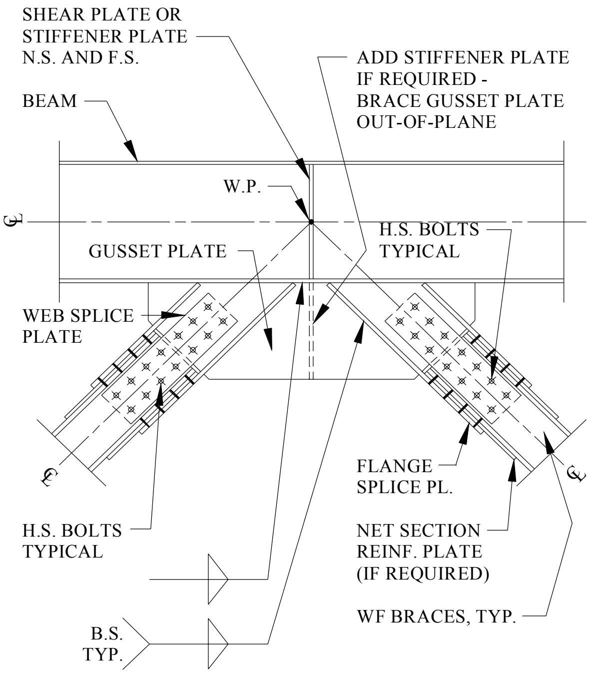


Figure 6-1f

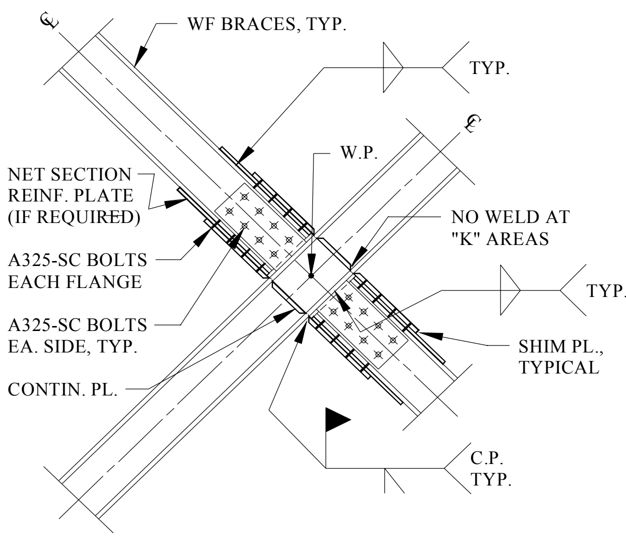


Figure 6-1g

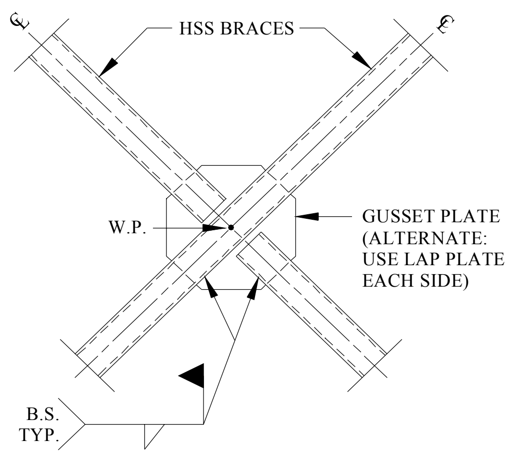
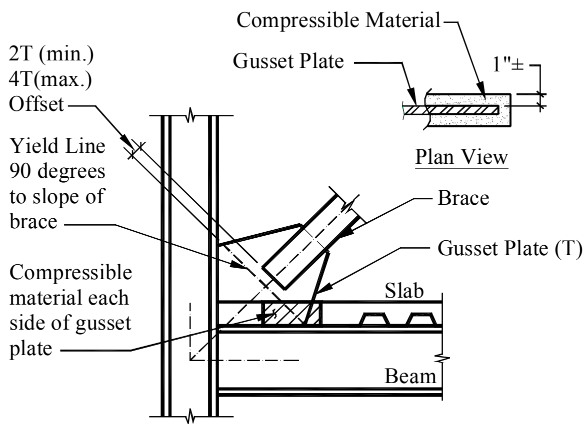


Figure 6-1h

Figures 6-1G and 6-1H show the bracing splices at a single-story X brace. For clarity of the connection in figure 6-1H, the net section reinforcement plates for the HSS sections have not been shown, but would be required when a single knife plate gusset is used, and the same 2T off-set requirements would also be required for the spliced brace from the continuous brace.

The design of the bracing connections can be impacted by many factors such as configuration, material strength and code requirements. Connection buckling behavior is also influenced by any floor system restraints such as any concrete slab or foundation confinement around the connection. The influence of the concrete slab confinement on the gusset plate cannot be ignored, but the influence will depend more on which elements are being confined (see figure 6-2).



Gusset Plate Yield Line Isolated from Surrounding Slab

Figure 6-2

6.1.1 Gusset Plates Designed for Out-Of-Plane Buckling

The gusset plate can be isolated from the concrete-filled metal deck slab by providing a 1x wood shim, sytrofoam, or other material on each side of the gusset plate that is removed after the concrete hardens. The length of the isolated area along each side of the gusset plate only needs to extend from the end of the gusset plate to beyond the portion of yield line, which occurs below the slab. The gap can then be filled with a flexible caulking or safing for fireproofing. The buckling length of the gusset plate is still determined using the Whitmore section (refer to the December 1998 *Steel TIPS*, "Seismic Behavior and Design of Gusset Plates" by Abolhassan Astaneh-Asl for a detailed discussion of the Whitmore section).

A stiffener plate equal to the depth of the slab can be welded to the end of the gusset plate on the beam. This moves the restraint point for out-of-plane buckling of the gusset plate up to the top of the slab (the gusset plate is proportioned so the yield line crosses the edge of gusset plate at or above the slab surface). This stiffener plate is not mandatory if the gusset plate is designed for the yield line to occur above the slab and the gusset plate free edge length doesn't require stiffeners. The addition of the stiffener in this case only helps assure the designer as to where the gusset plate yield line will occur.

The buckling line of the gusset plate must be perpendicular to the axis of the brace. The code requires the brace stop not less than 2t from this buckling line (yield line) where "t" is the gusset plate thickness. Because each beam and bay size will most likely be different, this will impact the dimensioning of the gusset plate to assure the gusset plate buckling yield line remains perpendicular to the axis of the brace. This causes additional shop detailing since all connections may be slightly different, which would therefore encourage the use of braced frames which buckle in-plane. Gusset plate stiffeners may be required to ensure the required out-of-plane buckling behavior about a gusset plate yield line perpendicular to the axis of the brace.

The engineer is encouraged to use repetition of the gusset plate if designing for out-of-plane by using constant bay widths.

6.1.2 Gusset Plates Designed for In-Plane Buckling

The concrete fill over the metal deck will most likely prevent the gusset plate from buckling below the top of the concrete. Conservatively, the recommended gusset plate buckling length should still be measured from the end of the brace to the top of the steel beam as determined using the Whitmore section. No special gusset plate detailing due to concrete confinement is required.

The authors recommend ignoring the floor slab thickness to prevent buckling of the gusset plate when designing the gusset plate. This is based upon the fact that the floor slab is generally rather thin. If the designer decides to reduce the gusset plate buckling length to take advantage of the slab stiffening effects, it would still be wise to consider that the slab fixity occurs one inch or so below the top of the concrete to allow for some surface concrete spalling.

The other consideration is the width of concrete slab perpendicular to the gusset plate providing the confinement. Brace frames located along the perimeter of the building typically have a limited concrete slab width between the exterior face of the gusset plate and the perimeter of the building slab edge form. The designer must determine if there is sufficient concrete strength to provide the buckling confinement of the gusset plate. The designer should also consider the reduced shear strength of lightweight concrete compared to normal weight concrete.

If both the gusset plate and brace are embedded in the concrete, as often can occur at the foundation, there will most likely be no buckling by the gusset plate. The suggested embedded brace design length is still measured from connection work-point to connection work-point. No reduction in brace length due to shallow concrete confinement is recommended, especially since the typical brace/gusset plate embedment in the concrete is generally less than 18 inches and it would likely be difficult to develop fixity.

If the brace embedment in the concrete is deep enough, and the brace is detailed to develop its flexural capacity ($1.1R_yF_yZ$), then a shorter brace length measured from upper connection work-point to a depth not less than the brace width (example: eight inches for an HSS 8x8), below the concrete surface would seem appropriate. The design engineer could conservatively use K=1.0 for evaluating the brace or determine the appropriate K value. The foundation end of a deeply embedded brace is likely fixed in-plane and out-of-plane.

6.1.3 Gusset Plate Strength

The gusset plate should be designed to resist buckling based upon using not less than the brace nominal buckling load of $P_n=(F_{cr}A_g)$. Note that minimum design force P_n is not reduced by ϕ_c. The brace buckling capacity F_{cr} should be based on the actual brace length in lieu of the traditional work-point to work-point length commonly used in analysis, see figure 6-3.

In computer analyses, the brace is typically sized based upon the brace diagonal length being from connection work-point to connection work-point. The actual brace length will have significantly higher buckling capacity than what the brace was actually assumed to have based upon the longer design (work-point to work-point) length. Using the actual brace length is considered to be a capacity design approach and the gusset plate should be designed to have design compression strength greater than the assumed brace buckling capacity.

In short braces, the designer may want to consider modifying $P_n=(F_{cr}A_g)$ to include R_y in determining F_{cr}. In long slender braces, R_y doesn't have much impact since the steel yield strength becomes less important in compression buckling due to the length, shape and section properties of the brace. The longer slender braces are expected to buckle early and the gusset plate is detailed to accommodate the brace buckling direction. Also, the gusset plate design may be more dependent upon the brace tensile force $A_gF_yR_y$. The authors suggest always including R_y in determining F_{cr} as a conservative measure for determining the compression axial force for gusset plate buckling design.

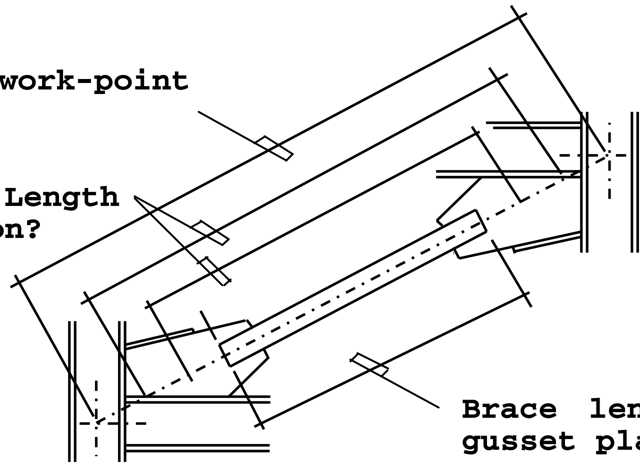


Figure 6-3 Brace Design Length

In short braces, the actual steel yield strength becomes an important consideration for compression design, especially if R_y is relatively large, such as R_y = 1.3 for HSS sections. The difference in member compression capacity can easily be seen in ASD column design comparing wide flange sections using F_y = 36 ksi and F_y = 50 ksi for short columns (50/36 = 1.4 increase). The additional short brace axial capacity is important to consider in determining the gusset plate axial design force to prevent buckling.

A shorter actual brace length could be used for the brace frame design than going from work-point to work-point, but the design length recommended by the authors should not be less than the actual brace length plus the diagonal distance along the gusset plate to the face of column or beam at each end of the brace. Using the gusset plate hinge point to gusset plate hinge point as the brace design length to determine the brace size is not recommended by the authors. Part of reluctance by the authors to use this shorter design length is based upon the fact that a similar building constructed ten years ago would have a significantly larger brace size than the same building today using the shorter gusset plate hinge point to gusset plate hinge point brace design length. The new shorter design length will lead to a smaller, more slender brace member, which will buckle sooner than a larger brace member would that was designed by the work-point to work-point philosophy from ten years ago, but was actually installed to the shorter length (hinge point to hinge point).

The result is that the older brace frame building would likely remain stiffer and elastic longer under seismic loading due to the larger brace size than would a new building designed today with its smaller brace size. The question that occurs is, are we confident in designing less stiff brace frame buildings than in prior years that will have less reserve strength since smaller braces are being used? By using the longer brace design length as stated above (but still shorter than the work-point to work-point design length philosophy), a stiffer building design is being encouraged. Also, because of uncertainties in the actual inelastic action/behavior in the brace framing system the authors recommend not to further shorten the brace design length by subtracting for the gusset plate lengths to the end of the brace member.

Using the actual gusset plate hinge point to hinge point brace length for brace compression design is permissible by the building code and the selected brace compression design length is left up to the designer.

6.1.4 Gusset Plate "K" Value for Plate Buckling

The "K" value used for the buckling design of the gusset plate should conservatively be 1.2 for out-of-plane bending and 0.80 may be used for in-plane buckling until further research is completed. The gusset plate unbraced length L_g used for KL/r calculations should be taken along the centerline of the brace (see figures B-6, B-7). An average $L_g = (L_g1 + L_g2 + L_g3)/3$ can also be used as shown in figure B-6A. These lengths (L_g1, L_g2, L_g3) are measured from the center and each end of the Whitmore Section width. Note that because of brace slope, the average L_g length may be more than the centerline length L_g2. The average L_g is used in the design example in part B. Also see section 4.3.c in Professor Abolhassan Astaneh-Asl's *Steel TIPS*, "Seismic Behavior and Design of Gusset Plates."

6.2 Gusset Plate Detailing Considerations

6.2.1 Gusset Plate Thickness and Shape

Due to practical dimensioning requirements of the gusset plate edges (rounding of length dimensions to ¼ or ½ inch increments) and steel erection tolerances, the minimum 2t offset from the yield line will often be slightly greater and may approach offset values of 3t or 4t.

At the brace end of the gusset plate, a minimum of one inch offset from the brace to the gusset plate sloped edge should be provided. The sloped angle measured away from the brace axis, starting from this edge, should not be less than 30 degrees, if possible. This is a practical minimum to maximize the Whitmore Section width for gusset plate compression strength.

Previously, the 30-degree minimum was also encouraged to help minimize gusset plate block shear when only the term $0.30F_u$ was used to determine the gusset plate shear strength along the entire block shear failure line. This is no longer the case today since both tension and shear components are typically included in the block shear calculation. Gusset plate block shear will never govern the slotted tube or pipe brace connection design if the gusset plate is designed to transfer the shear from the weld sizes resulting from developing the brace tensile capacity. The block shear in the gusset plate should still be checked for overall adequacy of the connection if the brace is not symmetrically centered on the gusset plate as shown in Figure B-7 (equal distance each side of brace centerline to gusset plate edge). If the brace is offset to one side of the gusset plate there is the possibility of a block shear rupture limit state profile occurring similar to that of a typical beam web connection where the flange has been coped (AISC LRFD Manual, figure C-J4.1). Block shear occurs along just two sides, instead of three sides, when the brace is offset on the gusset.

Similarly, shear rupture of the wall of the tube or pipe brace should not govern if the brace is checked for shear strength to develop the tensile capacity of the brace through the welds to the gusset plate. The HSS brace shear rupture capacity is calculated using the side wall thickness times the length of the brace slot overlap on the gusset plate determined by the required weld length to develop tensile capacity of the brace (see the design example calculations in part B).

The recommended minimum 30-degree angle slope along the gusset plate edge, after measuring over for the one-inch offset from the HSS brace, is not mandatory, and could be less than 30 degrees. The axial strength of the gusset plate depends upon the gusset plate effective width, thickness and unbraced length L_g. Using the 30-degree angle slope to determine the gusset plate width (Whitmore Section) will result in a thinner gusset plate when checking gusset plate compression buckling. If the slope is less than 30 degrees after offsetting the one inch from the HSS brace (example: at zero degrees you are using a rectangular plate), the gusset plate design width becomes less, and therefore the gusset plate must be thicker to provide enough cross-sectional area to resist the axial compression buckling force of the brace. The disadvantage of the thicker gusset plate is that so much of the tube or pipe brace cross-sectional area is removed when the slot for the gusset plate is cut into the brace, thereby requiring thicker reinforcement side plates to make up for the lost area. As the reinforcement plates become thicker, it can become more difficult to weld them to the HSS brace since there is limited flat-width area to weld the plates. The authors recommend that the designer stay with the 30-degree offset to help minimize gusset plate thickness.

As a rule of thumb, the gusset plate thickness should be in the range of twice the wall thickness of the brace tube and pipe sections, especially where the brace/gusset plate lap lengths are roughly equal to the brace width. As the lap length becomes longer, the gusset plate can be thinner. The lap length is dependent upon the shear lag/shear rupture capacity of the brace member and weld thickness used. The gusset plate lap length for tubes and pipes should be about twice the tube width or pipe diameter to minimize weld size and help reduce shear lag. Using gusset plates with thicknesses twice the wall thickness of the brace is a good starting point also for checking the axial buckling capacity of the gusset plate.

Example:	Brace HSS 8x8x5/8	Gusset plate: 1-1/4″ thick
	Brace HSS 6x6x1/2	Gusset plate: 1″ thick
	Brace HSS 5x5x3/8	Gusset plate: 3/4″ thick
	Brace HSS 4x4x1/4	Gusset plate: 1/2″ thick

For most cases, the gusset plate should be 1/2 inch thick minimum, and should probably always have a yield strength F_y = 50 ksi for larger braces.

Gusset plates made from F_y = 50 ksi steel are recommended by the authors for two reasons: first, for increased gusset plate tensile strength. Increased gusset plate tensile strength is important since R_y =1.3 for brace HSS sections (ASTM A500 steel with F_y=46 ksi) is an average for the actual yield strength, and it is not uncommon for HSS sections to have higher yield strengths in excess of 60 ksi (1.3 x 46 = 60 ksi). Using F_y = 50 ksi for gusset plates provides reserve tensile strength to account for the likelihood that some HSS sections will have higher yield strengths, since the steel fabricator is not in a position to reject HSS sections from the steel mill just because the actual HSS yield strength F_y is between 60 and 80 ksi. The authors admit that this is a conservative approach since not all braces in the building will develop tension yielding during an earthquake. The higher strength steel (50 ksi) used for gusset plates does not provide much additional buckling strength compared with using A36 steel (F_y=36 ksi), depending upon the gusset plate buckling length L_g.

Second, the higher strength steel reduces the gusset plate thickness required for connection tensile strength compared with using A36 steel.

Excessively thick gusset plates should be avoided. For smaller miscellaneous braces (HSS 3x3, 2-L3x3, etc.) that would typically be used for minor bracing, thinner plates such as 3/8 inch thick, or gusset plate yield strength of F_y=36 ksi are probably appropriate. The length of the brace overlap on the gusset plate will also impact the required yield strength (F_y) of the gusset plates. The longer the brace weld length to gusset plate, the more likely that gusset plate yield strength F_y = 36 ksi can be used. Also, A36 steel is more readily available for plate material than A572 Gr. 50.

As a good design practice, the authors recommend that the web thickness of the beam should not be less than 75 percent of the gusset plate thickness to help prevent web crippling of the beam web. When the brace slope is very shallow relative to the steel beam, the resulting vertical component of the brace tensile/compression force is small also; therefore, the 75 percent minimum web thickness would likely be overly conservative and not appropriate. In this case, the beam web should be checked for the design forces. In any case, web crippling of the beam needs to be checked and beam web stiffeners provided if necessary.

6.2.2 Connection Welds

The length of the brace weld to the gusset plate should generally not be less than twice the width of the brace’s least cross section dimension (example: HSS 6x6 use 12-inch long welds to gusset plate). This will help reduce brace shear lag and reduce shear stresses in the welds used in slotted tube brace connections. The length of lap of the brace over the gusset plate should allow for the termination of the weld away from the edge of the gusset plate as prescribed in the *D1.1: Structural Welding Code – Steel,* American Welding Society, Miami, FL, 2004 (AWS D1.1).

The gusset plate thickness should be in the range of twice the weld size required to attach the gusset plate. Actual thickness will be determined based upon the shear rupture of the gusset plate base metal and axial compression strength of the gusset plate. Based upon the shear rupture of the gusset at the weld and the shear rupture of the weld, the ratio of the thickness of the gusset plate relative to the weld thickness will be as follows:

Shear rupture strength fillet welds (70 ksi electrode):
$R_n = 0.6F_uA_{nv} = 0.6(70)(0.707)(A_{nv})(2 \text{ weld lines}) = 59.4(A_{nv})$
(Note that fillet welds occur on each side of the gusset plate; therefore, have two weld lines along each gusset plate shear plane)

Shear rupture strength of gusset plate:
A36 Plate (F_y=36): $R_n = 0.6F_uA_{nv} = 0.6(58)(A_{nv}) = 34.8(A_{nv})$
A572 Plate (F_y=50): $R_n = 0.6F_uA_{nv} = 0.6(65)(A_{nv}) = 39.0(A_{nv})$

Assume gusset plate rupture R_n = weld rupture R_n
Assume gusset plate length and weld length = unit length = 1″
Assume weld thickness = 1″, Find minimum gusset thickness t

F_y = 36 ksi:
t =(weld strength)(1x1)/(Gusset strength)(1)
=(59.4 x 1 x 1)/(34.8 x 1) = 1.7
∴ Minimum Gusset thickness 1.7 times weld thickness
F_y = 50 ksi:

t =(59.4 x 1 x 1)/(39.0 x 1) = 1.5

∴ Minimum Gusset thickness 1.5 times weld thickness

The actual gusset plate thickness may still need to be increased to satisfy the axial buckling requirements of the gusset plate.

The use of a gusset plate thickness that is twice the weld thickness will provide some flexibility with field tolerance regarding lap length of the brace over the gusset plate. A field condition that occurs frequently is when the brace is too long so the 2T offset from the yield line cannot be maintained. To fix this, the brace must be shortened in the field to maintain the 2T offset, which also results in shortening of the brace fillet weld lengths to gusset plate). A thicker gusset plate would allow for remaining fillet welds to be thickened and still not lead to shear/tension rupture of the gusset plate.

Fillet weld sizes will typically be in the range of 7/16 inch to 5/8 inch for the larger braces (example: HSS 6x6 and larger). In the field fix previously mentioned the brace wall thickness would have to be checked for shear rupture, based upon the thicker weld size to see if the brace was still adequate. Using an initial weld length one to two inches longer than required by design will also help provide some flexibility with field erection tolerances, especially when weld lengths have to be shortened and thickened as discussed in the example above.

The size of the slot in the tube or pipe should be 1/16 to 1/8 inch larger than the gusset plate thickness to facilitate erection of the brace. The weld size needs to be increased accordingly to compensate for the gap along the side of the gusset plate when the gap is larger than 1/16 inch, per AWS D1.1. This typically occurs when the slotted brace is held tight to one side of the gusset plate for welding leaving the 1/8-inch gap on the other side.

Because of the slotted brace connections used with gusset plates, the braces will typically need reinforcement plates to make up for the loss in net section (see design example, section B3.2.2). The welding of the reinforcement plate needs to be adequate to develop the tensile strength of the reinforcement plate. Consideration needs to be given to the fact that the reinforcement plates may be of different yield strength than the brace in determining the overall brace tension capacity.

6.2.3 Bolted Connections

The 1997 UBC and 2002 AISC *Seismic Provisions* (section 7.2) require all bolts to be pretensioned high strength with Class A or better faying surfaces. Joints don’t have to be designed as slip critical (example: A325SC); they can be designed as bearing (example: A325N values) and called-out/detailed as A325SC slip critical on the connection details to make sure the bolt is fully tensioned.

Oversize holes are not currently allowed except that short slotted holes are allowed if the slots are perpendicular to the line of force. In the authors' opinion, not allowing oversize holes will prevent field reaming that may be necessary to correct field fit-up problems. The authors hope that the 2005 AISC *Seismic Provisions* will allow for oversize holes. Tests have shown that slippage at bolted joints is not detrimental to the performance of braced frames and in fact adds some energy dissipation.

AISC *Seismic Provisions* section 7.2 also does not allow sharing of loads between bolts and welds in the same connection since slip critical bolts may slip under earthquake loading. Net section requirements will also likely require reinforcement plates to be added to the member to make up for the lost area due to the bolt holes.

6.2.4 Preventing Gusset Plate Buckling

The length of the unsupported edge of the gusset plate needs to be checked to prevent premature buckling. This is recommended by the authors since we are depending upon inelastic behavior in the gusset plate. The unsupported edge has historically been checked in truss design for elastic design loads. Since for seismic loads we are depending upon inelastic action, the allowable unsupported gusset plate edge length is reduced. The following equation is suggested:

$$(L_{eg}/t) < 0.75\sqrt{E_s/F_y} = 18.0'' \text{ for } F_y = 50 \text{ ksi}$$
$$= 21.2'' \text{ for } F_y = 36 \text{ ksi}$$

When the free edge length exceeds this dimension, either horizontal or vertical stiffener plates need to be added to the gusset plate. The location of the stiffener must be verified to ensure that it does not cross the yield line of the gusset plate. Depending upon brace slopes and where the yield line crosses the free edge, use of horizontal stiffener plates may be limited, allowing only for vertical stiffener plates (see figure B-8 in the design example in part B).

A special case can occur at the beam mid-span V or two-story X brace connection where the yield lines of two braces on a single gusset plate cross before intersecting the free edge because the brace slopes are very steep. In this particular case, the gusset plate must be coped back between the braces. The cope should be along a 30-degree slope and deep enough that yield lines now intersect the free edge of the gusset plate before crossing. Consideration needs to then be given to in-plane bending of the gusset plate at the reduced section. This gusset plate condition is usually the result of providing V or two-story X bracing in a relatively tall, narrow width framed bay which might be better suited as a single diagonal brace or single-story X brace if this bay must be used.

See the December 1998 *Steel TIPS*, "Seismic Behavior and Design of Gusset Plates," by Abolhassen Astaneh-Asl for a very good discussion and recommendations regarding gusset plate design and behavior.

6.2.5 Uniform Force Method

The Uniform Force Method as illustrated in the AISC LRFD Manual (3rd Ed., under "Bracing Connections") is used to determine beam/column gusset plate connection dimensions and interface forces only, and LRFD "Member Strength" properties are used to actually design the connection. The Whitmore Method is typically used to evaluate the compression buckling of the gusset plate.

The design efficiency of the Uniform Force Method is best when the centerlines of the column, beam and brace all meet at a point called the work-point (WP). Special cases of the Uniform Force Method are allowed for:

Special Case 1:	Alternative brace work-points (WP at corner of gusset plate).
Special Case 2:	Minimizing shear in the beam/column connections.
Special Case 3:	No gusset plate to column web connections.

All of these special cases can result in heavier connections. The reader is referred to the AISC LRFD Manual for a more in-depth review of the Uniform Force Method.

The Uniform Force Method can be difficult to use for SCBF framing if trying to eliminate moment at the gusset-to-beam or gusset-to-column connection. This is due to the predetermined geometry resulting from the minimum 2t offset required from the gusset plate yield line to the end of the brace and the requirement that the yield line be perpendicular to the brace axis. The gusset plate design may be more easily handled by considering it as an existing connection and accounting for the eccentricities ($\alpha'-\alpha$) and ($\beta'-\beta$) in the gusset-to-beam and gusset-to-column interfaces once the gusset plate has been proportioned to satisfy the yield line and 2t offset requirements.

If the gusset plate geometry allows Special Case 2 to be used, this would reduce the need to add web stiffeners to the beam, since the shear in the beam-column connection would be reduced, thus lowering fabrication costs. Special Case 2 is not always advantageous, as shown in the design example in part B. In the design example, the beam (W36) is relatively deep compared to the column depth. Thus, the gusset plate geometry required was such that to provide the minimum 2t yield zone causes the length of interface between gusset plate and the beam top flange to be longer than the interface between the gusset plate and the column. Since the gusset plate cannot be ideally proportioned so that α and β are at the interface centroids, the gusset plate is designed as if it were an existing connection with β set to equal β' (so no moment exists at the gusset plate-column flange interface) and α' is solved for using the Uniform Force Method equations in the LRFD Manual. Therefore it is important to realize that the beam depth, relative to the column depth, influences which Uniform Force Method should be used. Had a W24 beam section been selected (it would have been heavier than the W36 shown in the example) Uniform Force Method 2 would probably have worked.

Also see the 1998 *Steel TIPS*, "Seismic Behavior and Design of Gusset Plates" (section 4.4) for other methods that can be used to determine gusset plate support forces. The Uniform Force Method is just one method for designing the gusset plate connection and other vector mechanic methods are just as valid.

6.2.6 Beam-Column-Gusset Plate Intersection

The behavior of the beam-column-gusset plate intersection during an earthquake is complicated by the high forces generated by the braced frame in combination with building drift, which causes bending moments to occur at this location. The gusset plate acts like a large haunch trying to fix the beam to the column. Recent tests of EBF and combination of BRBF with moment frames have shown that damage and failures can occur at the gusset plate-column-beam connections. The BRBF systems are expected to have lower drifts as compared to SCBF, therefore more frame action is expected for SCBF which may be detrimental to the SCBF connections. In the authors' opinion, the current codes do not adequately address the behavior at this intersection. More research and testing is needed to verify the behavior at this crucial location in SCBF, EBF, and BRBF and to provide guidelines for designing the gusset plate connection to beam and column accounting for building drift (see section 9, "Summary of Research Needs").

Also, recent tests have shown that fracture can occur between the web stiffener and beam web. It would be prudent to hold back the fillet welds along the gusset plate interfaces at least two times the weld thickness from the beam-column intersection. This will help eliminate a triaxial stress condition at this location as well as eliminate an area that is hard to weld without creating flaws.

6.3 General Detailing Considerations

The typical beam-column connection will depend upon where the building is located in the United States. On the West Coast, more of the fabrication shops are set up for welding as opposed to the Midwest and East Coast where bolting is more common.

In the seismic areas the authors recommend using a single shear plate connection to the column instead of a double angle connection. This eliminates having to check prying action of the angles at the column.

The drafting of the bracing connection details on the structural drawings needs to include guidelines that allow the shop detailer some flexibility in actual dimensioning of the gusset plate and 2t minimum offset requirements. Framing member lengths on the shop drawings are typically detailed to 1/16 of an inch, so slight changes in architectural dimensions that occur during the design stages will change brace slopes which impact gusset plate dimensions. The authors recommend that the following information be included on the structural drawings for the shop detailer:

1. Which side of brace is the critical angle for gusset plate design: between column and brace or brace and beam? The critical angle is defined as the side of the brace where the gusset plate 2t offset yield line first intersects the column flange or beam flange causing gusset plate restraint. The other end of the yield line typically crosses the free edge of the gusset plate where there is no restraint. An explanation using the critical angle approach can be found in the presentation "Practical Design of Steel SCBF: An Alternative to Steel Moment Frames," (Flynn, L.; Cochran, M., November 1999) available from SEAOSC, Whittier, CA.

2. Provide only one gusset plate dimension, preferably on the side of the critical angle of the brace slope. Provide enough additional guideline information to allow the shop detailer to dimension the other edges of the gusset plate and maintain the 2t offset from the yield line. If you provide all of the gusset plate dimensions, and the detailer's dimensions do not match, then you will have to verify on the shop drawings which dimensions are correct.

3. Provide information regarding maximum gusset plate dimension adjustments (examples: 2t offset actually shall not exceed 4t; no change of more than 1 inch in actual gusset plate dimension shown on structural drawings allowed). Fabricator should note on shop drawings when limitations are exceeded.

4. Require that individual gusset plate details be drawn to scale on the shop drawings so that the yield line (2t to 4t offset) and stiffener plate location can be checked by scale.

5. Recommend that the steel detailer call the structural engineer to discuss the intent of the information shown on these details prior to beginning detailing.

6.4 Specific Detailing Considerations:

The following is a list of detailing considerations for each type of brace.

6.4.1 Angles

The net area requirements of UBC section 2213.8.3.2 or D1 of the LRFD specification will most likely preclude the use of F_y = 50 ksi angles with a bolted connection since the yield strength is close to the tensile strength (65 ksi). A36 steel will have a little more flexibility since its minimum yield is at 36 ksi versus minimum tensile strength of 58 ksi. If bolts are used, the angles will require plate reinforcement to meet net section requirements. For large axial forces, A490 bolts may be required for bolted gusset plate connections and bolted stitch plates between legs of built-up angle braces. If 50 ksi angles are used, welding of the angles to the gusset plates and stitch plates most likely would be required to develop the connections.

The angle size will have to be less than L8x for 50 ksi material in order to satisfy compact shape criteria.

If two rows of bolts are used, the spacing between bolts in a horizontal row most likely will have to be in the range of 4-1/2 inches on center (o.c.) in order to satisfy block shear requirements. Welding will probably be the more economical connection to use because of the number of bolts and potential field fit-up problems at gusset and stitch plates. When using an angle to gusset plate welded connection, the angle welds should be balanced to account for the eccentricity between the welds and the neutral axis of the angle shape.

The allowable b/t and Kl/r ratios greatly limit the number of available angle sizes to use. Typically the brace lengths will have to be both short to comply with Kl/r restrictions and have thick legs for b/t restrictions.

Double or quad angle braces can be oriented using unequal leg angles such that the brace member buckles in-plane or out-of-plane. By using the angle long legs horizontal (LLH) or outstanding (LLO), the brace can be made to buckle in-plane. Double angles with unequal legs should not be oriented with long legs back to back since the difference between the built up brace member r_x and r_y is very small and assuring brace buckling in-plane is not reliable. The authors recommend that r_x/r_y ratio not be more than 0.65 when the brace is to buckle in-plane.

6.4.2 Tubes (Rectangular HSS Sections)

The use of rectangular tubes turned flat for bending about the minor axis is encouraged since this will promote in-plane buckling as opposed to out-of-plane buckling of the brace. In-plane bending of the brace allows for the brace to be brought closer into the connection. The gusset plate must still be designed to resist out-of-plane buckling, but the 2t yield line offset perpendicular to the brace axis need not be considered.

When steel stud infill walls are provided above and below the diagonal brace member, a gap between the diagonal brace and the steel stud infill walls above and below the diagonal brace should be detailed to help assure in-plane buckling of the brace.

Flat plates can be added to the tubes that do not meet the b/t limitations, but these will have to be high strength plates with a minimum yield of not less than 50 ksi. The reinforcement of the tube section may be more practical than using a wide flange section as an alternative.

6.4.3 Pipes and Round HSS

Pipes are difficult to slot cut at each end and have the slots align so that there is no rotation or angle offset between the slots. Slight variations in the slot alignment on the pipe make it difficult to slide the pipe on to the attached brace frame gusset plates.

Use of pipe braces does not allow you to control in-plane and out-of-plane buckling based upon the brace member. The direction of brace buckling can only be influenced by gusset plate orientation. The gusset plate oriented vertically (traditional connection: gusset framing between column and beam flanges, and gusset buckles out-of-plane) or turned horizontal (gusset welds only to column flange or beam flange, and gusset buckles in-plane of frame).

6.4.4 Wide Flange Shapes

When using the Uniform Force Method for the gusset plate design, the weld lengths along the column and beam do not have to be the same length as the gusset plate dimensions. The important thing is to make sure that these two weld lines are centered on the centroid of the gusset plate based upon determining the α and β dimensions for the Uniform Force Method. Similarly, this is done for the bolted connection of the gusset plate to the column flange. Limiting the weld lengths to less than the gusset plate length is probably more academic than practical in the shop or field. The welder will likely weld the entire length of the gusset plate, thereby changing the expected performance of the brace connection. The authors would not recommend this practice of limiting the weld lengths on the gusset plate, and recommend that the design consider the entire length of the gusset plate being welded.

6.4.4.1 Wide Flange Brace Weak Axis Buckling

Wide flange diagonal brace members can be oriented about their weak axes to encourage brace buckling in-plane of the brace frame instead of out-of-plane. The wide flange column is oriented so the wide flange diagonal brace member frames into the weak axis of the column (the column web). The wide flange brace member is oriented about its weak axis (the wide flange member is turned so its web is in a flat position), which allows side lap plates on each side of the column to be connected directly from brace flange to column flange. If the column and brace wide flange sections are not the same depth, welded shim plates are required to build out either the brace or column depth so the side lap plates can be fit up flat. The brace frame beam framing into the column web can also be oriented about its weak axis (beam web in horizontal position). This allows a single side lap plate on each side of the column to interconnect the beam, brace and column flanges together. The June 1988 *Steel TIPS,* "Seismic Design Practice For Steel Buildings," shows a brace frame design example using wide flange brace members oriented in the weak direction. The design engineer is reminded that this design example would have to be updated for the current building code requirements.

The wide flange beam oriented about its weak axis may not be practical for supporting the floor-framing members. Also the shear tab for any beams framing into the column flanges would have to attach to the lap plate instead of directly to the column flange.

7.0 SCBF FOUNDATION CONSIDERATIONS

A number of issues need to be considered when designing foundations under SCBF. Some of these are listed below:

7.1 General

Is the foundation capable of resisting brace forces, assuming that compression braces have buckled and the tension braces are at the yielded state, for both downward and uplift forces? The brace frame foundations (spread footings, pile caps, etc.) for the individual columns of the brace frame should be tied together to provide a load transfer path for the brace frame lateral forces to the foundation. Options include:

- A reinforced concrete grade beam can be provided to tie the individual spread footings together.

- A concrete encased steel beam can be used to tie the base of the brace frame columns together.

- Pile caps require tie beams between individual pile caps that can be utilized to transfer lateral forces to the foundation.

These grade beams/tie beams can be doweled to the slab-on-grade to help disburse the lateral forces into the building foundation. If the brace frame column spacing is close, combined footings can be utilized for both resisting overturning and tying the column bases together. Both continuous combined footings and individual spread footings connected together using concrete grade beams are illustrated in the figures in the part B example problem.

When concrete grade beams are used, some engineering offices have welded continuous A706 rebar between the column base plates to physically tie the steel columns together and provide a load path to transfer the lateral forces to the foundation. Since these rebar are typically located in the slab-on-grade, U-shaped rebar dowels are inserted over these rebar and into the concrete grade beam to tie the slab/grade beam system together and encapsulate these rebar.

7.1.1 Uplift Anchor Rods

The large column uplift forces at the foundation are generally transferred into the concrete foundations by anchor rods attached to the sides of the column or through the column base plate. Due to the large uplift forces these anchor rods should be of a high strength material such as ASTM A449, ASTM A193 Grade B7, or the relatively new ASTM 1554, to help reduce the size (diameter) and quantity of anchor rods required.

The embedment of the anchor rods into the concrete should be deep enough to develop the yield strength of the uplift anchor rods, not just the uplift design forces. Thick steel bearing plates (washer plates) nutted onto the end of the anchor rods provide the uplift bearing for the embedded anchor rods. As a rule of thumb, the thickness of these bearing plates should not be less than 40 percent of the width of the bearing plate (example: 4″x4″ washer plate minimum thickness 1.6″) and may need to be thicker based upon calculation. Some engineers will tie several anchor rods together using a single bearing plate instead of individual bearing plates for each anchor rod.

The concrete shear cone is calculated to determine the minimum required embedment depth to develop the anchor rod strength. Since there are multiple anchor rods for each base plate, the individual anchor rod shear cones generally overlap each other, so a single larger truncated shear cone encompassing all of the uplift anchors is calculated. Often the footing thickness is significantly deeper than the minimum required anchor rod embedment since the footing weight is being used to resist the uplift forces. The authors' recommend that the anchor rods extend to within 12 inches of the bottom of the footing or pile cap to engage the entire concrete footing mass, not just the concrete mass above the minimum anchor rod embedment depths. Additional flexural (footing top rebar reinforcing mats) and shear (stirrups) reinforcement may be required to develop the strength of the footing in addition to the required footing bottom reinforcing mat.

7.2 Shallow Spread Footing Foundations

Is there enough footing weight to resist forces due to tension brace yielding? If not, should foundation rocking be considered as the limit state for the SCBF frames? How is the foundation rocking analyzed and what impact does it have on the design of the braces? For example, a rocking foundation may not allow the braces to reach their tension capacity. Also consider tying footings together and adding drilled piers under the SCBF column footings to resist uplift. If rocking is to be utilized, the connection of the brace frame to the foundation must be capable of lifting the entire foundation associated with that column (including footing weight, slab on top of footing, grade beams tied to footings, and inverted triangular soil wedges along all vertical sides of the footing that will act with the footing). A factor of safety should be applied to the calculated amount of foundation dead load uplift resistance to account for additional uplift resistance due to dynamic response during an earthquake. The authors recommend that the brace connection to the foundation should not be designed for less than 1.3 times the calculated foundation dead load.

7.3 Drilled Pier and Pile Foundations

Piles or piers have soil friction resistance and can be designed to resist SCBF uplift forces; therefore, the engineer should be able to achieve tension yielding in the braces. Lateral forces in piers are resisted by passive pressure against the piers and pier caps. Slab-soil friction may not be available if the soil settles from under the slab-on-grade and pier caps. For pile foundations lateral forces are resisted by pile flexure and shear. It is important to understand and design for the soil/pile interaction. The slab-on-grade should be designed and detailed to distribute the overall foundation lateral forces so that most or all of the piles participate in the lateral resistance. Bent rebar "L" or "U" dowels should be provided between the pile caps and slab-on-grade. Pile caps may need to have vertical shear reinforcement to transfer the uplift loads from the steel columns, through the pile caps, to the piles. When there is uplift on the pile cap, there is both vertical tension and horizontal shear occurring across a horizontal plane in the pile cap.

7.4 Subterranean Structures (Basements)

Subterranean structures usually provide a large mass (structural slabs, foundations and walls) for resisting uplift so that brace tensile yielding should be able to be achieved in the SCBF. Connections to the subterranean structure should be designed for the tensile yielding of the brace. Subterranean structures have large wall/footing areas to resist lateral forces using passive soil resistance as well as slab-on-grade and foundation sliding friction resistance if shallow wall footings are used. Sliding friction may not be available if the subterranean structure is pile supported.

8.0 CONSTRUCTION

8.1 Erection Considerations

A frequent problem with diagonal bracing erection is attempting to erect the brace after the beam above has been placed for closely spaced columns. Typically the brace length needs to be reduced for tube sections so that the brace can be slid over the knife gusset plates at each end. This results in longer gusset plates for the connection. This is typically a problem when the brace is brought all the way into the connection. When using the 2t offset from the brace to the perpendicular yield line, this should be less of a problem.

One solution when the bracing is brought all the way into the connection (brace buckling-in-plane) is to erect and temporarily shore the braces in place prior to placing the floor beam above. The beam with knife plate is then dropped in place and the diagonal tubes are then swung up into place.

Another solution is to field weld one or both end gusset plates after the brace is erected, with gusset plate(s) attached to the brace with erection bolts.

Column splices should be placed at the lower end of the middle one-third of the column clear height between floors (Per AISC *Seismic Provisions,* section 13.5, the column splice must occur within the middle one-third of the column clear height). This will facilitate welding by allowing the welder to stand on the floor. Also see section 2.4 above regarding column splices.

8.2 Structural Observation

Just like moment frame connections, the brace frame connections need to be inspected to make sure that they comply with the detailing requirements shown on the drawings. These are additional checks beyond the normal welding inspection requirements.

8.2.1 2T Minimum Offset

The brace frame connection needs to be inspected to make sure that the minimum 2t offset from the yield line has been maintained (special inspection). During erection the slotted brace may be slipped too far onto the gusset plate at one end of the brace providing less than the required 2t offset. This should be checked just prior to welding and again after welding. The reason to check again after welding is that sometimes the brace has been knocked out of alignment after initial fit-up but not rechecked prior to welding. If the 2t offset is not maintained, then a field fix will be required to establish the 2t minimum offset. The authors recommend design and detailing for a 3t offset to help with field erection and to help minimize the impact of field errors.

8.2.2 Gusset Plate Minimum Edge Distance

The brace is usually detailed to be centered on the end of the gusset plate (equal distance from centerline of brace to gusset edge, each side of the brace). The gusset plate detailing discussed earlier recommends a minimum one-inch offset from the face of the brace to the edge of the gusset plate. The minimum offset requirements need to be checked to make sure the block shear/tension rupture capacity of the gusset plate has not been reduced. If the brace is located too close to one edge, possible block shear rip-out from gusset plate can occur along two failure edges (tension, and one shear edge, AISC LRFD Manual, figure C-J4.1) instead of three failure lines (tension, and two shear edge, AISC LRFD Manual, figure C-J4.2).

When brace erection aids are not used (erection bolts, clip angles), braces are tack welded in plate, or gusset plate shapes are incorrectly fabricated relative to each other at each end of the brace, the erected brace may not end up centered on the gusset plates.

8.2.3 Gusset Plate Yield Line Restraint Locations

If the gusset plate yield line is to be isolated from the concrete floor slab (to prevent restraint by the slab), then this must be checked. This includes verifying that any temporary forms used to create the void pockets have been removed. Stiffener plates should be checked to make sure they don’t cross the yield line as well.

8.2.4 HSS Reinforcement Plate Location on Brace

The reinforcement plate location on the sides of the HSS brace should be checked. The reinforcement plate is to be located so as to develop the strength of the reinforcement plate at each end of the oversized slot length (for erection purposes) in the HSS brace. To develop the strength of the reinforcement plate, the plate must extend each end beyond the length of the open slot extending beyond the end of the gusset plate. Often the brace's erection slot may be made longer in the shop or field than what is detailed on the shop drawings, or simply incorrectly located in the shop relative to the slot length in the brace. If the reinforcing plate is mislocated, or does not extend far enough to make up for the oversized slot length, then the HSS net section may be inadequate at the connection. The authors recommend that the length of the slot beyond the end of the gusset be limited to (HSS nominal width/2) and be detailed as such on the drawings.

9.0 SUMMARY OF RESEARCH NEEDS

There are still a number of areas in brace frame design that require additional research. The authors feel that the following issues are important and hope that they will be addressed in the future.

9.1 Gusset Plates

The behavior at the intersection of the beam-column-gusset plate intersection during an earthquake is complicated due to the high forces generated by the braced frame in combination with building drift, which causes bending moments to occur at this location. The gusset plate acts like a large haunch trying to fix the beam to the column. Recent tests of EBF and combination of BRBF with moment frames have shown that damage and failures can occur at the gusset plate-column-beam connections. When the rectangular-shaped brace frame bay distorts horizontally, due to lateral loading, into a parallelogram shape (beams horizontal, columns leaning) the tension brace gusset plates have been observed to undergo compression buckling due to the initial orthogonal angle between the beam and column becoming acute (less than 90 degrees). When the beam-column joint becomes less than a 90-degree angle, a compression strut forms in the gusset plate, causing out-of-plane buckling and tearing of the gusset plate due to the tension brace pulling on the gusset plate. In the authors' opinion, the current codes do not adequately address the behavior at this intersection. More research and testing is needed to verify the behavior at this crucial location in SCBF, EBF and BRBF and to provide guidelines for designing the gusset plate connection to beam and column accounting for building drift.

9.2 Beams in V and Inverted V (Chevron) Brace Systems

Refer to section 4.4, V and Inverted V (Chevron) Braces, for a discussion of beam-end fixity. In conjunction with the behavior of gusset plates discussed in section 9.1, bending at the ends of beams in V brace systems have been generally assumed to be "pinned-pinned". This is a very conservative assumption and leads to a large beam size. In reality, the gusset plates at the end of the beam (for the braces that go up to the next level) provide fixity at the ends of the beams. More research is needed to determine the behavior of the beam and gusset plates for this condition and guidelines need to be established so the designer can properly size the beams and end connections, including the gusset plates.

9.3 In-Plane Buckling of Braces

More guidance is needed for designing braces that buckle in the plane of the braced frame, assuming the gusset plates are vertical. The end moments in the brace are potentially much higher than in cases where the brace buckles out-of-plane and plastic moments occur in the gusset plates instead of in the brace. For the in-plane buckling condition, the brace-end connections must be strong and ductile enough so that hinges can form in the brace itself, and not in the gusset plate.

More guidance is also needed when braces are spliced onto horizontal gusset plates, which in turn are spliced onto vertical gusset plates, so that when the horizontal plate buckles out-of-plane, the brace is actually buckling in the plane of the frame.

9.4 Brace Frame Base Plates

Guidance and testing is also needed in the design of the base plate to transfer the brace yielding tension forces and column uplift forces into the foundation through the base plate. Developing the greater of the brace tensile capacity or column uplift forces (resulting from frame overturning moments), into the foundation can result in very large uplift loads with limited attachment points to transfer these forces from the gusset plate and column into the base plate. Wide flange columns have a very limited flange width (b_f) and web depth (d) to extend more than four or six anchor rods total up the sides of the column for attachment using wing plates (example: bolts in tension transfer forces by shear into the column flange using bearing plates and gusset plates attached to the side of the column, see AISC LRFD Manual, figure 11-15(b), Base Plates for Uplift).

What is the actual base plate stress distribution as the uplift forces are transferred through the base plate from the wide flange column (plan view = H shape) and gusset plate (plan view = line shape) to the anchor rods? Are complete penetration welds required, or are fillet welds satisfactory to attach the column to the base plate when the column uplift forces can be 100 kips or greater? How should the anchor rods be laid out on the base plate? Are there group pattern effects where the engineer can't assume the tension force is equal in all anchor bolts? If the anchor bolt distance is less than twice the thickness of the base plate from the column or gusset plate then shear may govern base plate thickness. Shearing failure of a thick brace frame column base plate was observed after the 1994 Northridge Earthquake. If the anchor bolt is located more than twice the base plate thickness away from the column or gusset plate then bending is the governing factor. The further the anchor bolts are away from the column and gusset plate, the larger the bending forces and resulting thicker base plate. Since the uplift loads are so large, additional guidance/research is needed to determine the best ways to transfer these large uplift forces into the foundation.

Where does most of the horizontal shear transfer from the brace frame to the foundation occur?

- Into the foundation, at shear lugs under the column base plate? And what is the best way to design these shear lugs?

- Or, through bearing of the concrete slab-on-grade on the surface of the column flange above the base plate when the brace connection is buried below the top of the concrete slab-on-grade?

If most of the shear transfer is through bearing onto the concrete slab-on-grade, than additional slab reinforcing should be provided around the brace frame columns and to tie the concrete slab-on-grade and column footing together.

9.5 R_f Factor

In the steel materials currently available for buildings, the actual material tensile yield strength exceeds the minimum specified ASTM yield strength for a given steel grade and therefore a R_y factor has been introduced so the brace connections are not under-designed. Since the yield F_y for steel is actually greater than assumed, it is likely that the actual F_u for steel is also greater. AISC is currently conducting research to determine what this R_f factor should be, if any is required. Until this research is completed, R_f is to be taken as 1.0. R_f will have an impact on the shear lag calculation for the brace member.

9.6 OCBF and Tensile Capacity of Braces

Depending upon the type of OCBF structure being designed, the authors question whether the connections need to develop the actual tensile capacity of the brace. In lightweight OCBF structures this may be particularly true, where very long braces resisting relatively small compression loads must be designed for the tensile yield capacity of the brace. The resulting OCBF brace connection design forces may greatly exceed the building base shear (V) of the structure by a magnitude of two times, three times, or more.

Example buildings:

- Buildings with steel girders and beams, wood joist infilled framed floors and roofs covered with plywood sheathing. Curtain wall perimeter or steel studs with stucco.
- Research/manufacturing hi-bay buildings with bare metal deck roofs (no concrete fill) and perimeter walls with girts and metal siding.

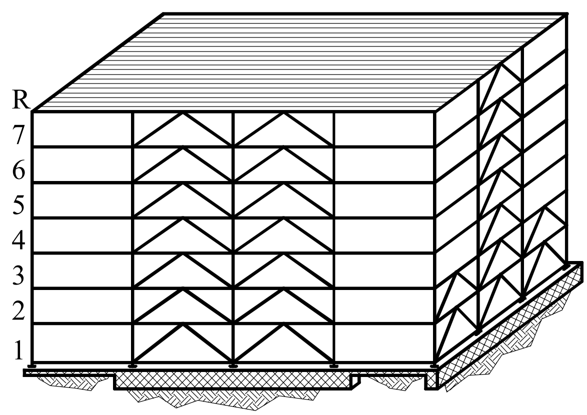
Additional research appears appropriate to determine if designing the OCBF's connections based upon a building system factor of R=1 might be more appropriate, a force level typically significantly greater than (Ω_o x calculated design force), but less than $A_gF_yR_y$.

PART B: DESIGN EXAMPLE USING TUBE (HSS) BRACES

B1.0 INTRODUCTION

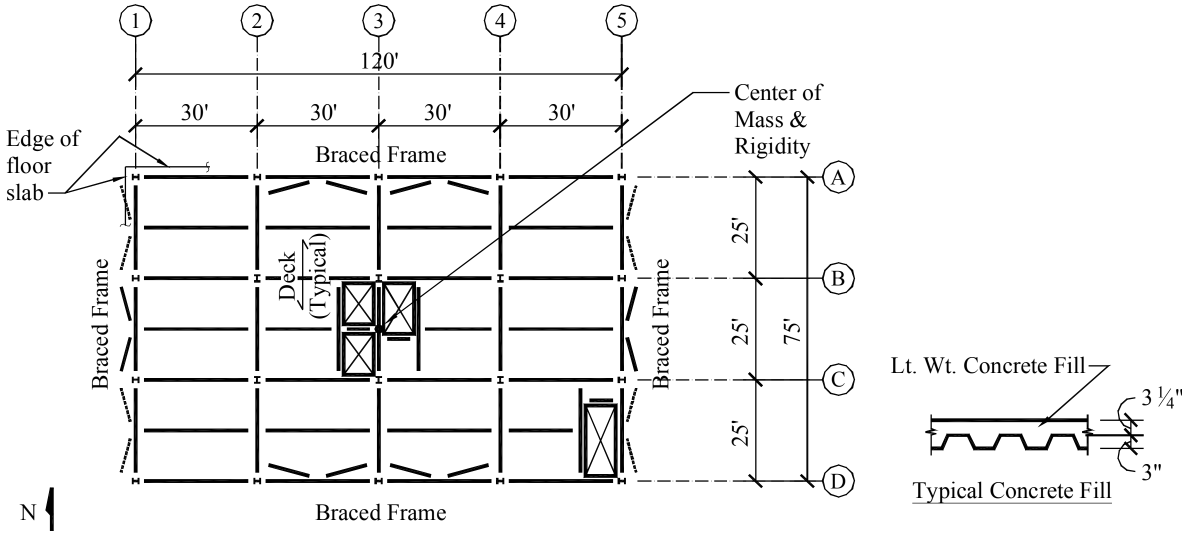
B1.1 Preface and Building Elevations

This design example is a seven-story building with SCBF in both directions and is the same example building that was presented in the AISC Seminar, "Seismic Design and Detailing of Braced Frame Structures," by Lanny J. Flynn, in 2000 (see figures B-1 and B-2). The LRFD method is used as explained in section 1.2. The loading follows the LRFD load combinations in the 1997 UBC and the design and detailing generally follows the current 2002 AISC *Seismic Provisions*.



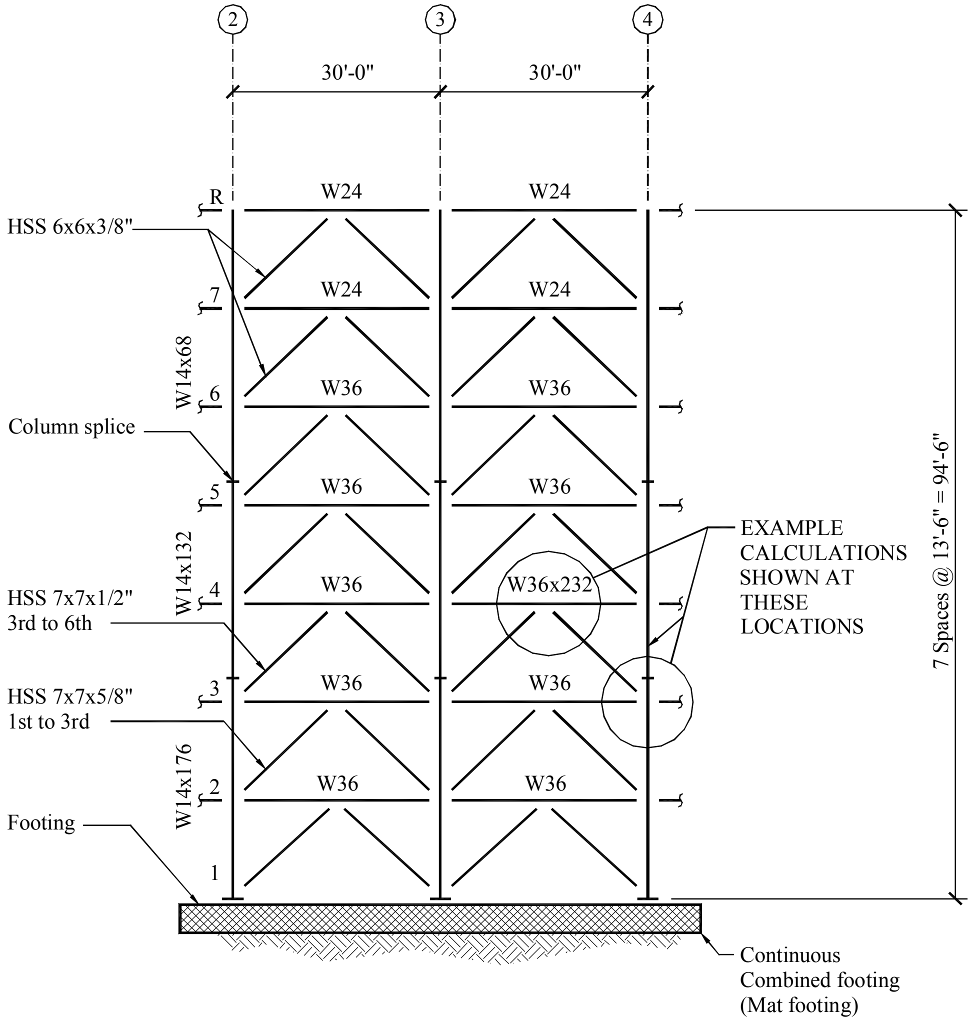
Seven Story Building

Figure B-1



Typical Floor Plan

Figure B-2



SCBF Elevation (N & S) Lines A & D

Figure B-3A

A continuous combined footing (mat footing) is utilized in figure B-3A to resist overturning forces, since the overturning width of the braced frame is only two bays. If the net uplift forces are too large, then friction piles may be required to provide uplift resistance. A combination of spread footings and grade beams are utilized in figure B-3B to resist overturning since the overturning forces can be distributed over three brace frame bays.

A positive tie between the braced frame columns bases (example: continuous rebar welded from base plate to base plate, steel beams with headed shear studs framed between columns buried in the foundation) should be provided to tie the brace frames together as well. This tie provides a better load path to transfer lateral forces into the foundation.

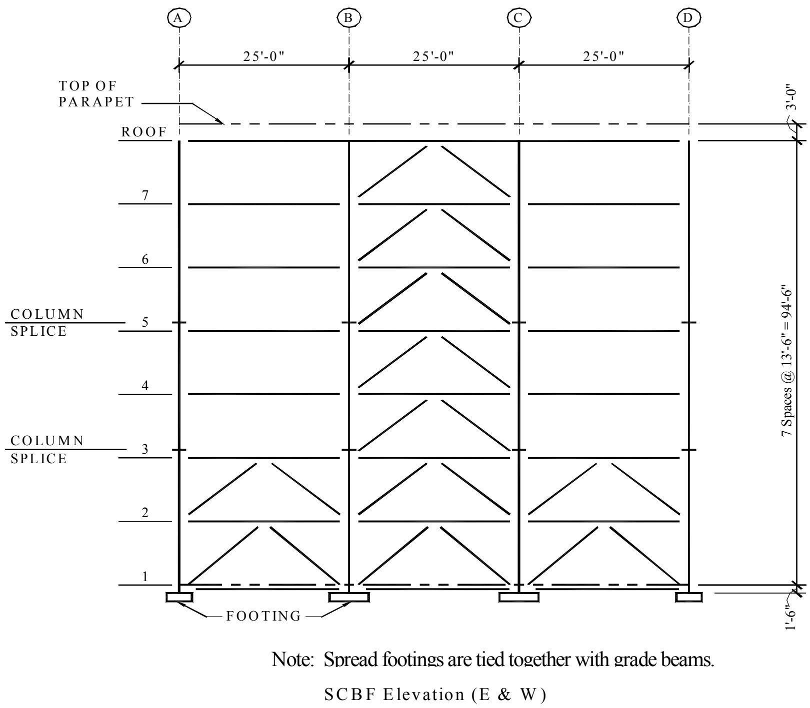


Figure B-3B

B1.2 Seismic Forces

B1.2.1 Building Site

The seismic design of bracing members will be in accordance with division IV, chapter 22 of the 1997 edition of the UBC and the 2002 AISC *Seismic Provisions*. Building lateral forces are calculated based upon LRFD load combinations in UBC section 1612.2.2 and the base shear equations in section 1630.2. The building is located in Seismic Zone No. 4 with the following site conditions:

a. $Z = 0.4$ (Table 16-I)
b. Soil Profile Type = S_D (Table 16-J)
c. $I = 1.0$ (Table 16-K)
d. Seismic Source Type = B (Table 16-U)
e. $C_a = 0.44\ N_a$ (Table 16-Q)
f. Near Field Factor $N_a = 1.3$ (Table 16-S)
(seismic source type = B, building located less than 2 km from a fault)
g. Near Field velocity Factor $N_v = 1.6$ (Table 16-T)
h. $R_w = 6.4$ (Table 16-N)

The seismic forces used in the bracing example problems are based upon a "regular shaped" building as shown in figure B-2. In reality, if this building were defined as being irregular, then a dynamic analysis would be required for structures over five stories or 65 feet tall.

B1.2.2 Building Configuration

Figure B-2 is the plan view of the building.
Figures B-3A and B-3B illustrate the braced frame elevations.
Building footprint: 122'-6" x 77'-6"
Building height: 7 stories
Typical floor height: 13'-6"
Perimeter steel frames:

East-west direction:	2 bays braced
North-south direction:	1 bay braced at upper floor stepping to three bays at the first and second floors.

B1.2.3 Material Specifications

Steel brace frame:

Wide flange:	ASTM A992 (replaces ASTM A572, grade 50, enhanced per Technical Bulletin #3)
Rectangular HSS sections:	ASTM A500 grade B (F_y=46 ksi)
Plates:	ASTM A36 or ASTM A572, grade 50
Welding electrodes:	E70XX
High strength bolts:	1" diameter A325N with standard holes for beam to beam connections. Use A325SC, Class A for beam to column connections and at gusset and brace connections
Brace frame column anchor rods at base plate:	ASTM A1554 (Grade 105)
Alternates:	ASTM A449, ASTM A193 Grade B7

The 1997 UBC does not require material overstrength factors to be used to estimate the expected steel yield. However, as with steel moment frames, the actual yield strength of steel brace frame members is significantly higher than the specified yield strength and should be considered with material overstrength factors. The large steel overstrength yields are the result of how domestic steel is being produced in the United States.

The design engineer is encouraged to review other model building/design codes that incorporate material overstrength factors, such as the 2002 AISC *Seismic Provisions for Structural Steel Buildings*, and to utilize these factors in their design. The design example in this *Steel TIPS* includes the use of material overstrength factors "R_y".

Table B-1 summarizes the current recommendations of overstrength multipliers to use in steel design that are similar to the 2002 AISC *Seismic Provisions*.

Steel Shape	**ASTM**	**Recommended Overstrength Factor "R_y"**
Wide flange	A36	1.5
	A572, Grade 50 A992, Grade 50	1.1
Rectangular tubes (HSS sections)	A500, Grade B, F_y = 46 ksi	1.3
Round tubes (HSS sections)	A500, Grade B, F_y = 42 ksi	1.3
Pipes	A53, Grade B, F_y = 35 ksi	1.4
Angles	A36	1.5
	A572	1.1
Plate	A36 A572	1.1

Table B-1 Recommended Overstrength Factors

The overstrength factor R_y = 1.0 is conservatively used for steel plates in the design example since they are typically used for connections and yielding is to occur in the brace before it occurs in the connection. By underestimating the gusset plate actual yield strength, thicker plates will be required to resist the design forces.

A review of steel yield strengths suggests a high likelihood that current domestic steel production methods using scrap steel will result in average yields not less than 55 ksi for most shapes (49 ksi for pipe). Some imported steel of ASTM A36 and A572 though may have lower actual yield strengths.

B1.2.4 Typical Gravity Loads

The gravity loads are the same as the seven-story building used in the previous *Steel TIPS*, "Seismic Design of Special Concentrically Braced Frames" (1995), and are repeated below.

Roof Loading:

Roofing and Insulation	7.0 psf
Metal Deck	3.0
Concrete Fill	44.0
Ceiling and Mechanical	5.0

Steel framing and fireproofing	8.0
Total Dead Load	67.0 psf

Roof Live Load (Reducible):

UBC section 1607	20.0 psf
Total Roof Load	87.0 psf

Floor Loading:

Metal Deck	3.0 psf
Concrete fill	44.0
Ceiling and Mechanical	5.0
Partitions (UBC 1606.2)	20.0
Steel framing(beams, columns (and spray-on fireproofing)	13.0
Total Dead Load	85.0 psf
Floor Live Load (reducible): UBC section 1607.3	50.0 psf
Total Floor Load	135.0 psf

Curtain Wall:

Average weight including columns and spandrel covers	15.0 psf

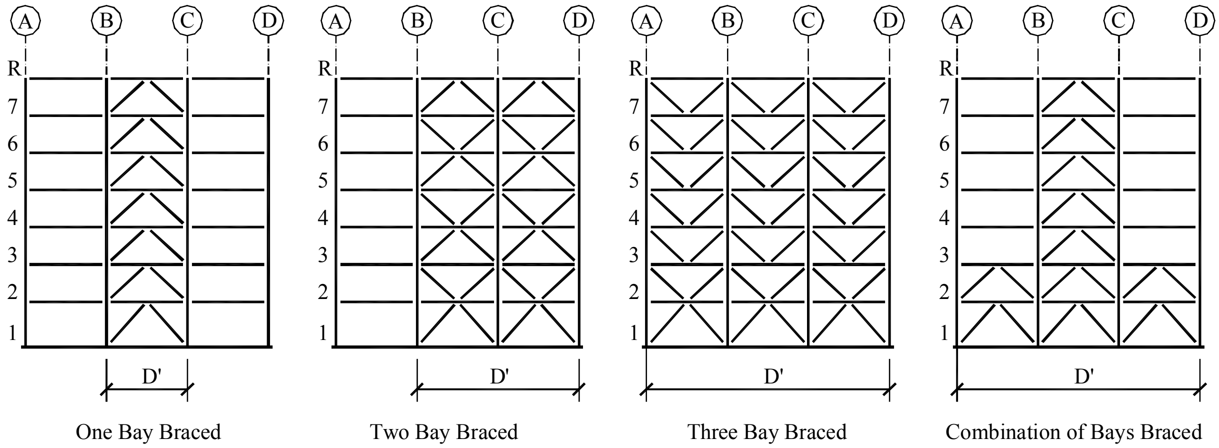
1. Notes:

   a. It is very common in buildings to be designed for a live load of 80 psf. The 50 psf is a carryover from the previous *Steel TIPS*.

   b. The floor-to-floor height has been increased to 13'-6" since this is the approximate typical height which has been used in recent years and allows the ductwork to pass under the beams.

B1.2.5 Seismic Base Shear

Figure B-4 illustrates some alternate bracing schemes that could be considered in lieu of the bracing scheme selected for the design example. The brace sizes selected for these alternate schemes would likely have different cross-sectional areas from the brace sizes chosen for this design example. The resulting building periods and member forces would vary between these alternate bracing schemes and the Design Example. As can be seen below, the calculated building period is governed by Method B as opposed to Method A.



Possible Brace Configurations

Figure B-4

1997 UBC Earthquake Load E:

$E = \rho E_h + E_v$ ρ = redundancy factor

$E_h = V$ $E_v = .5C_aID$ (UBC Eq. 30-1)

1997 UBC Base Shear Equations "V":

$V = C_v\ I\ W\ /(R\ T)$

Seismic in the y Direction (North-South):

$C_v = .64N_v = (.64)(1.6) = 1.02$ (UBC table 16R)

$I = 1.0$, $W = 5933.8$ kips, $R=6.4$

$T_a = C_t(h_a)^{3/4} = .02(94.5)^{3/4} = 0.606$ sec.
(UBC 1630.2.2 Method A)

$T_b = 0.734$ seconds $< (1.3)(.606) = 0.787$ seconds
(UBC 1630.2.2 Method B)

$V = (1.02)(1)(W)/(6.4)(0.734) = 0.217W$

Seismic in the x Direction (East-West):

$C_v = (.64)(1.6) = 1.02$ $I = 1.0$ $W = 5933.8$ kips

$R = 6.4$

$T_a = C_t\ (h_a)^{3/4} = 0.606$ seconds

$T_b = 0.641$ seconds $< (1.3)(.606) = 0.787$ seconds

$V = (1.02)(1)(W)/(6.4)(0.641) = 0.249W$

Additional Base Shear Checks:

$V_{max} = 2.5\ C_a\ I\ W\ /R$

$C_a = .44N_a = (.44)(1.3) = 0.57$

$V = (2.5)(0.57)(1)(W)/6.4 = 0.223W < 0.249W$ (E-W)
$> 0.217W$ (N-S)

$V_{min} > 0.11\ C_a\ I\ W = (0.11)(0.57)(1)(W) = 0.063W$ (UBC Eq. 30-6)

$V_{min} > 0.8\ Z\ N_v\ I\ W/R = (0.8)(0.4)(1.6)(1)(W)/6.4 = 0.080W$ (UBC Eq. 30-7)

Base Shear V:

$V_x = 0.223W = 1326$ kips $E_h = 1326$ kips (E-W)
$V_y = 0.217W = 1294$ kips $E_h = 1294$ kips (N-S)

Check Redundancy (ρ): (UBC 1630.1.1)

Building Floor Area: 122.5’ x 77.5’ = 9494 sq. ft.

Story Shear:
(At braces below 5th floor = 2/3 height of building)
V_x (below 5th) = 1326 - 50 - 100 - 150 = 1026 kips
V_y (below 5th) = 1294 - 46 - 92 - 138 = 1018 kips
V_y (below 3rd) = 1248 kips

Perimeter Braced Frame Stiffness:

East-West: Assume equal braced frame stiffness for both
North-South: Assume equal braced frame stiffness for both.

Accounting for five percent mass displacement and torsion, maximum force in the brace line exceeds 50 percent of story shear, approximately equal to 55 percent for this particular design example.

Below 5th Floor:
$r_{max-x} = [(0.55)(1026)]/[(4\text{ braces})(1026)] = 0.1375$
$r_{max-y} = [(0.55)(1018)]/[(2\text{ braces})(1018)] = 0.2750$

Below 3rd Floor:
r_{max-y} = [(0.55)(1248)]/[(6 braces)(1248)] = 0.0917

$\rho_x = 2-[20/(0.1375\sqrt{9494})] = 0.51 < 1.0$

$\rho_y = 2-[20/(0.2750\sqrt{9494})] = 1.25 > 1.00 \;\; < 1.50$

Per 1997 UBC section 1629.4.2, since the redundancy factor is equal to 1.0 in the east-west direction, soil type is S_D, and there are no specific structural irregularities (building is regular), C_a can be reduced from 1.3 to 1.1. The building's maximum base shear requirement could then be reduced from 0.223W to 0.189W in this direction. This design example assumes the building is regular, but conservatively still uses the higher base shear of 0.223W for illustration purposes.

In the north-south direction, since the redundancy factor is greater than 1.0, the base shear now becomes (1.25)(0.217W). The redundancy factor is based upon the maximum element-story shear ratio (r_{max}) of any brace in the building. Even though the r_i value at the first and second floor braces results in a redundancy factor less than 1.0, the 1.25 must be applied at all floor levels of the building in that direction.

B1.2.6 Building Story Forces

Whiplash effect check (F_t) at top of building:

F_t (East-West) = 0.0 kips (T < 0.7 Sec.)

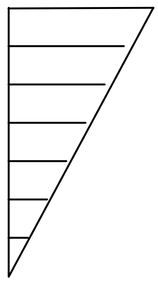
F_t (North-South) = .07 TV = 66.5 kips (T > 0.7 Sec.)
(UBC Eq. 30-14)

The following table B-2 summarizes the distribution of seismic force at each floor level of the building per the 1997 UBC.

Story Forces					**East-West**		**North-South**	
Floor Level	h_x (feet)	w_x (kips)	$w_x h_x$	$\frac{w_x h_x}{\sum w_i h_i}$	F_x (kips)	V_x (kips)	F_y (kips)	V_y (kips)
Roof	94.5	699	66,055	.211	279	-	406	-
7	81.0	872	70,656	.226	299	279	346	406
6	67.5	872	58,880	.188	249	578	289	752
5	54.0	872	47,104	.150	199	827	231	1041
4	40.5	872	35,328	.113	150	1026	173	1272
3	27.0	872	23,552	.075	100	1176	115	1445
2	13.5	872	11,726	.037	50	1276	58	1560
1	-	-	-	-	-	1326	-	1618
Sum	-	5933	313352	1.000	1326	-	1618	-

Table B-2 Distribution of Seismic Force at each Floor Level of Building

Building Story Forces (including redundancy):



	F_x	X Direction (East-West)	Y Direction (North-South)
Roof		279 kips	325 x 1.25 = 406 kips
7		299 kips	277 x 1.25 = 346 kips
6		249 kips	231 x 1.25 = 289 kips
5		199 kips	185 x 1.25 = 231 kips
4		150 kips	138 x 1.25 = 173 kips
3		100 kips	92 x 1.25 = 115 kips
2		50 kips	46 x 1.25 = 58 kips
Total:		V_x = 1326 kips	V_y = 1618 kips

East-West Diaphragm Design Forces (section 1633.2.9):

F_p Roof Min. = 199 kips F_p Roof Max = 398 kips

F_p Floor Min. = 249 kips F_p Floor Max = 497 kips

Level	Force (kips)	Sum. Force	Weight (kips)	Sum. Weight	Force/ Weight	F_p (kips)
R	279	279	699	699	0.4	280
7	299	578	872.3	1571.3	0.368	321
6	249	827	872.3	2443.6	0.338	295
5	199	1026	872.3	3315.9	0.309	270
4	150	1176	872.3	4188.2	0.281	249(min)
3	100	1276	872.3	5060.5	0.252	249(min)
2	50	1326	872.3	5933.8	0.224	249(min)

Table B-3 Summary of Seismic Force Distribution

Note that at the second, third, and fourth floor levels minimum code forces govern design of the floor diaphragm.

Diaphragm Drag Forces (East-West):

Roof = 280/(75x120) = 31.2 psf
7 = 321/(75x120) = 35.6 psf
6 = 295/(75x120) = 32.8 psf
5 = 270/(75x120) = 30.0 psf
4,3,2 = 249/(75x120) = 27.7 psf

When a building has a symmetrical framing layout and stiffness, then using an average psf for determining diaphragm drag forces should be appropriate. When the framing layout is unsymmetrical or has significant differences in stiffness between frames, it would be more appropriate to distribute drag forces based upon frame rigidity.

B1.2.7 Building Drift

Results from the computer run:

Roof Drift Δs(East-West) = 1.61 inches
Δs(North-South) = 2.47 inches

The redundancy factor (ρ) was taken as equal to 1.0 for calculating drift and displacements (UBC 1630.1.1).

Per UBC 1630.10.2, check that story drifts do not exceed limit:

$\Delta_{mroof} < .025(h_{story})(N_{stories}) = .025(13.5' \times 12'')(7) = 28.0''$

Per UBC 1630.9.2, $\Delta_m = .7R\Delta_s = .7(6.4)(2.47) = 11'' < 28''$ Okay

B2.0 SCBF DESIGN

The remainder of this design example will concentrate on the E-W SCBF on lines A and D and will show calculations for the beam at level 4 and the column, braces and end connections between levels 3 and 4 to illustrate the design provisions using the LRFD method. See figure B-3A.

Square tube (HSS) braces were picked for this example because they are commonly used for brace sections for low-rise buildings in California. Other brace sections are possible such as pipes, wide flange sections and built-up multiple angles. Low cycle fatigue performance of the brace members was not considered to be important for this particular office building example.

The square tube HSS brace sizes were repeated at several floor levels in this example as opposed to being different sizes at each floor level of the building. Changing the brace sizes at each floor level results in a better inelastic distribution of forces between braces and floor levels during an earthquake. Repeating brace sizes at each floor level decreases both engineering office design time and steel fabrication time through repetition. The brace frame design performance decreases, though, since the brace member forces are greatest at the lowest floor level of the multiple floor level grouping with the same brace size.

Proportioning of the brace member sizes versus efficiency of design and fabrication are up to the individual designer to determine based upon their particular project needs.

The seven-story building in this example was modeled using the computer program "RamSteel." HSS sections were used in the model for the braces and W36 beam sections at the SCBF frames. The member forces shown in the following calculations are taken for the computer-run results based on the loading calculations presented above. The computer results indicated a significant redistribution of gravity forces in the braces when compared to a single bay V (chevron) SCBF.

Sizing the braced frame members can be affected by how the frames are modeled in the computer considering floor diaphragm attachments. Two cases were considered:

1. Floor diaphragm is attached at the beam mid-span brace connection.
2. Floor diaphragm is detached from the beam at mid-span.

The member forces shown below are based on case 2.

B2.1 SCBF Brace Design

B2.1.1 Design Forces

LRFD Basic Load Combinations:

$1.4D$	(UBC Eq. 12-1)
$1.2D + 1.6L + .5L_r$	(UBC Eq. 12-2)
$1.2D + 1.0E + .5L + .2S$	(UBC Eq. 12-5)
$.9D \ ^{+/}_{-}\ 1.0E$	(UBC Eq. 12-6)
$E = \rho E_h + E_v$	(UBC Eq. 30-1)

Forces in braces between the third and fourth floors from computer run:

Axial Dead Load	P_d = 25.8 kips
Axial Live Load	P_l = 11 kips
Axial Roof Live Load	P_{rl}= 1 kip
Axial Seismic Load	$P_e = 208 + E_v = 208 + .5C_aID$
	$= 208 + .5(.572)(25.8) = 215$ kips

Compression in braces (UBC equation 12-5, governs):

$$P_{uc} = (1.2 \times 25.8) + (1.0 \times 215) + (.5 \times 11) = 252 \text{ kips}$$

Tension in braces (UBC equation 12-6):

$$P_{ut} = (.9 \times 25.8) - (1.0 \times 215) = -192 \text{ kips}$$

B2.1.2 Brace Member Selection

Tube brace steel type A500 Grade B, specified F_y = 46 ksi

Try HSS 7x7x1/2 (t actual = 0.465″ thick = 0.93 t nominal):

b/t: < than or equal to $110/\sqrt{F_y}$ = 16.2 (1997 UBC)

< than or equal to $0.64\sqrt{E_s/F_y}$ = 16.06 (AISC 2002)

1997 UBC b/t: b/t = 7/0.5 = 14 < 16.2 okay

AISC b/t flat width:
b/t = (7-(3 x 0.465))/(.465) = 12.05 < 16.06 okay

(Note: Instead of doing the b/t calculation, the HSS brace member b/t ratios can be taken directly from either the AISC HSS Connections Manual or the AISC LRFD Manual, 3rd Ed.)

Kl/r < than or equal to $1000/\sqrt{F_y}$ = $1000/\sqrt{46}$ = 147

Kl/r = (1.0)(20.18)(12)/2.63 = 92 < 147 okay (UBC)

(Note; for AISC: Kl/r < $5.87\sqrt{E_s/F_y}$ = 147.4)

Check HSS 7x7x1/2 in compression:

(Note: The work-point to work-point length is 20.18 feet. Brace length from face of W36x beam to face of W36x beam is about 16 feet, and actual brace length, from end of brace to end of brace is approximately 13 feet.)

For the brace design, use the 16-foot length.

$\lambda_c = KL\sqrt{F_y/E}\ /r\pi$ (Eq. E2-4)

$\lambda_c = [(1.0)(16)(12)/2.63\pi]\sqrt{46/29000} = .9255 < 1.5$

Use LRFD equation E2-2

$\varphi F_{cr} = \varphi\left(0.658^{\lambda_c^2}\right)F_y$ $\lambda_c^2 = (.9255)^2 = .8565$

$\varphi F_{cr} = .85(.658)^{.8565}(46) = 27.3$ ksi

$\varphi P_n = \varphi F_{cr}A = (27.3)(11.6) = 316 > P_{uc} = 252$ kips okay.

Check HSS 7x7x1/2 in tension:

Okay by inspection, since axial compression governs minimum selected brace size ($P_{uc} > P_{ut}$), and not tension.

Use HSS7x7x1/2 brace

B2.2 SCBF Column Design (Column Between Levels Three and Four)

B2.2.1 Design Forces

When $P_u/\varphi P_n$ is greater than .4, the design must meet following requirements (AISC *Seismic Provisions* 8.3 & UBC division IV, section 2210):

Required axial compression strength determined from following load combinations from UBC 2210:

$$1.2P_d + .5P_l + \Omega_o P_e$$

$$.9P_d +/- \Omega_o P_e$$

and the basic LRFD load equations in UBC 1612.2.1

Column loads between levels three and four from the computer run:

P_d = 214 kips; P_l = 36 kips; P_{lr} = 9 kips; $E_v = 0.5C_aID$;

ρE_h = 315.6 kips

$$P_e = \rho E_H + E_v = 315.6 + (0.5)(0.572)(1)(214) = 377 \text{ kips}$$

The first two load combinations control by inspection since they include Ω_o and column force $P_u/\varphi P_n$ is greater than .4:

$$P_{uc} = 1.2(214) + .5(36) + 2.2(377) = 1104 \text{ kips}$$

Note:
1. Ω_o = 2.2 from UBC and Ω_o = 2.0 in IBC and ASCE-7. We have kept Ω_o = 2.2 in this example since the force levels are based on the UBC.)
2. ρ is not required in this load combination, but has been included for simplicity since column forces are taken from the computer program.

$$P_{ut} = .9(214) - 2.2(377) = -637 \text{ kips}$$

B2.2.2 Column Member Selection

Per AISC *Seismic Provisions* 13.5 (UBC, division IV, 2210) width-thickness ratios of stiffened and unstiffened compression elements of columns must meet requirements in AISC *Seismic Provisions* 13.2d (table I-8-1):

Try W14x132 (ASTM A992, F_y = 50 ksi)

Web h/t_w = 14.66/.645 = 22.7 < $253/\sqrt{F_y}$ = 35.8 okay

Flange $b/2t_f$ =14.73/[2(1.03)]=7.14 < $52/\sqrt{F_y}$ =7.3 okay

(Note: In AISC, 2002, b/2tf < $.30\sqrt{E_s/F_y}$

K = 1.0; L = 13.5 feet; Kl/r_y = (1.0)(13.5)(12)/3.67 = 44.1

$\lambda_c = [44.1/\pi]\sqrt{50/29000} = 0.583 \qquad \lambda_c^2 = 0.34$

$\varphi F_{cr} = .85(.658^{.34})(50) = 36.9$ ksi

φF_{cr} A = 36.9(38.8) = 1432 > 1104 kips okay <u>Use W14x132</u>

Since this is a braced frame the columns should be checked per LRFD Manual, chapter C for Second Order Effects (elastic analysis). The moment magnification factor for <u>*lateral translation*</u> "B2" is considered included in the computer analysis member forces output since the computer model included P-delta effects on the framing members. B2 can therefore be taken as equal to one. If the computer model does not include the building mass to determine the P-Delta effects, then B2 should be calculated. The moment magnification factor for <u>*no lateral translation*</u> "B1" is applicable to framing member dead loads, live loads, and seismic loads. Since the brace frame beams and braces in the computer model were modeled as pinned-ended members, there are no column bending moments and the product of B1 x M_{nt} equals zero, therefore no need to calculate B1.

The design engineer should give some consideration to the fact that the actual connection will have some fixity due to welding of braces, beams, columns and gusset plates together and might want to assume some bending moment in the column and beams as part of their design and include the magnification factor B1. If the beams and braces had been modeled as fixed end, then B1 would be calculated for the columns and applied to the column bending moments from the computer output.

B2.3 SCBF Beam Design (Beam at Level Four)

B2.3.1 Design Forces

Per AISC *Seismic Provisions*, 13.4a (UBC division IV, 2210) for V (chevron) braces:

1. Beam must be continuous between columns when intersected by braces.

2. Design beam to support tributary gravity loads assuming bracing not present.

   LRFD load combinations:

   $1.4D$ (UBC Eq 12-1)
   $1.2D + 1.6L + .5L_r$ (UBC Eq 12-2)
   $1.2D + .5L + .5L_r$ (UBC Eq 12-3)

3. Design beam to resist effects of LRFD load combinations for gravity, except the term "E" is replaced with Q_b, the maximum unbalanced vertical load effect applied to the beam by the braces. Q_b is calculated using P_y for brace in tension and $.3\varphi P_n$ for brace in compression.

The load combinations to be checked: (Controls)

$1.2D + .5L + Q_b$ (UBC 2213.9.4.1)

$0.9D - Q_b$ (UBC 2213.9.4.1)

Q_b: For a HSS7x7x1/2 brace
(Use HSS Manual or LRFD Manual, 3rd Ed.: $Ag = 11.6\ in^2$)

$P_y = P_t = R_y A_g F_y = (1.3)(11.6)(46) = 694$ kips
(R_y required by AISC Provisions 13.4a(3))

$P_c = .3\varphi P_n = (.3)(316) = 95$ kips

Brace vertical force components:

Brace length (work-point to work-point) = 20.18 feet

Story height = 13.5 feet, half bay width = 30/2 = 15 feet

Tension brace $P_{tv} = 694(13.5/20.18) = 464$ kips

Compression brace $P_{cv} = 95(13.5/20.18) = 64$ kips

$Q_b = 464 - 64 = 400$ kips

Load Combination: $1.2D + .5L + Q_b$ (governs)

Beam Bending moment (M_u):

$$M_u = (WL^2/8) + Q_bL/4$$

$$W_D = [(12.5/2 + 1.25)(85)] + 13.5(15) = 840 \text{ lbs/ft}$$

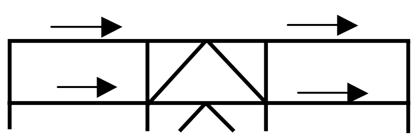
$$W_L = [(12.5/2 + (1.25-0.5)](50) = 350 \text{ lbs/ft}$$

$$M_u = M_{max} = [(1.2)(.84) + (.5)(.35)](30^2)/8 + 400(30)/4$$
$$= 133 + 3000 = 3133 \text{ ft-kips}$$

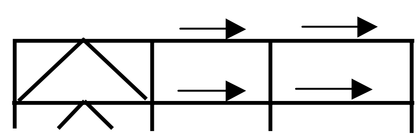
$$V_u = V_{max} = [(1.2)(0.84)+(0.5)(0.35)](15) + 400/2$$
$$= 17.7 + 200 = 218 \text{ kips}$$

Beam Axial Forces:

The brace framed beam axial forces are mainly seismic and based upon the symmetrical braced frame locations shown along the building perimeter grid lines in this design example. The seismic axial forces would be assumed to be applied equally from each end of the beam: half the beam is in compression and half the beam is in tension. If the braced frame layout were not symmetrical along the grid line, then the axial load applied at each end of the braced-frame beam would not be equal. The condition where all seismic forces would come from one side only occurs when the diaphragm dragline frames to the braced frame from one side only (example: single braced frame bay occurs at the corner of the building).



Symmetrical layout



Unsymmetrical layout

If the earthquake forces are large enough so that braces begin buckling during load reversals (compression, tension, compression, tension, etc.), an elastic redistribution of seismic forces would occur between braced frames resulting in a redistribution of forces in the dragline beams.

When designing the braced frame beam member for a single-bay braced frame, the seismic axial forces can be *conservatively* assumed to be coming from one side of the frame only, assuming that brace buckling and inelastic redistribution of seismic axial forces has occurred within the frame members. The beam axial force should be the sum of the horizontal components from both the yielded tension brace ($A_gF_yR_y$) and 30 percent of the compression brace critical buckling force ($0.3\varphi P_n$). Even when the compression brace begins to buckle, it is still resisting an axial load.

The assumption of all seismic forces coming from one end is more important when there are individual single-bay braced-frame bays (example: East and West frame elevations, upper floor levels) as opposed to multi-bay braced frames (example: North and South frame elevations), which help with redistribution of the seismic forces when a brace buckles. When multi-bay braced frames occur symmetrically along the gridline, then the assumption of seismic axial forces occurring equally from each end of the beam is easier to rationalize. When all bays along the gridline are braced (example: East and West Frame Elevations, at second- and third-floor levels) then assuming the seismic axial force occurs equally from both beam ends would seem sufficient for beam design. This assumption of how the axial load is applied can have a significant impact on the size of the beam, as shown below.

Axial load (P_u) in beam due to horizontal component of the tension brace force:

Seismic axial force from one side only:

$P_u = (P_t + .3\varphi_c P_n)(15/20.18) = (694 + .3(316))(15)/(20.18) = 586$ kips

Seismic axial force from both sides equally:

$P_u = P_t + .3\varphi_c P_n)(15/20.18)/2 = (694 + 95)(15)/(2 \times 20.18) = 293$ kips

Since this is a braced frame, and buckling of the braces will induce bending in the beam, LRFD Manual chapter C is checked for Second Order Effects (elastic analysis). The moment magnification factor for *lateral translation* B2 is not applicable since the beam flexural forces are determined from brace member capacity analysis as opposed to applied seismic forces from the computer model. The moment magnification factor for *no lateral translation* B1 is applicable to framing member dead loads, live loads, and seismic loads (example: load combination D+L+E).

The beam bending moment (M_x) is amplified for design purposes by the factor B1. Since the beam size is not known yet, an assumption of ten percent is made for B1 to help estimate the required beam size.

Assume B1 = 1.10, therefore $M_u = 1.10(3133) = 3446$ ft-kips

$Z_{req'd}$ approx. = $12M_u/F_y$ = (12)(3446)/50 = 827 in^3

Note: When selecting the beam, pick at least one size bigger than required by $Z_{req'd}$ to help account for axial load in the beam.

B2.3.2 Beam Member Selection

Try W36 x 232 S_x = 809 in^3; Z_x = 936 > 827 in^3 okay, A = 68.1 in^2, r_x = 14.8 in, r_y = 2.62 in, Ix = 15000 in^4

Beam unbraced length: X axis = 30 feet
Beam unbraced length: Y axis = 15 feet(mid-span brace only)
Beam unbraced length: Y axis= 7.5 feet(quarter point bracing)

Beam top and bottom flange assumed braced out-of-plane at quarter points and mid-span.

Kl/r_x =(30 x 12)/14.8 = 24.3

Kl/r_y =(15 x 12)/2.62 = 68.7 (beam braced at mid-span only)

Kl/r_y =(7.5 x 12)/2.62 = 34.4 (beam braced at quarter points)

Note: Top and bottom lateral braces shall have sufficient strength for .02 $F_y b_f t_{bf}$ of beam, per AISC 13.4g(4).

Second Order Effects (AISC LRFD Manual, 3rd Ed., chapter C):

$B1 = C_m/(1-P_u/P_{el}) \geq 1.0$ (AISC LRFD C1-2)

Note: Cm is conservatively taken as 1.0. It would actually be less, see LRFD Manual, chapter C

From AISC LRFD Manual, 1st Ed., chapter C:

$P_{el} = A_g F_y / \lambda_c^2$

$\lambda_c = KL \sqrt{F_y/E} / (r_x)(\pi) = 0.321 \quad \lambda_c^2 = 0.103$

P_{el} = (68.1)(50)/0.103 = 33,058 kips

From AISC LRFD Manual, 2nd and 3rd Ed., chapter C:

$P_{el} = \pi^2 EI/(KL)^2 = (3.14)^2(29000)(15000)/[(1.0)(30x12'')]^2$
$= 33,093$ kips

B1 = 1/[1 - (586/33,058)] = 1.018 < 1.10 assumed - Okay
B1 = 1/[1 - (586/33,093)] = 1.018 < 1.10 assumed - Okay

M_{ux} = 1.018 (3133) = 3190 ft-kips

Check beam interaction equation:

Since the beam axial load is so large at 586 kips, top and bottom beam flange bracing will be provided at the beam quarter points.

L_b = 30/4 = 7.5 ft.; L_p = 9.25 ft.> 7.5 ft. okay

Beam weak axis buckling length to ¼ points governs:

Kl/r_y = 7.5x12/2.62 = 34.4 > 24.3 = Kl/r_x = 30x12/14.8

$\varphi P_n = \varphi \left(0.658^{\lambda_c^2}\right) (F_y)(A_g)$,

$\lambda_c = Kl\sqrt{F_y/E})/(r_y)(\pi) = 0.4547$

$\lambda_c^2 = 0.207 \qquad \varphi P_n = (.85)(.658^{.207})(50)(68.1) = 2654$ kips

$M_{nx} = M_{px} = F_y R_y Z$ O $1.5M_y$ (AISC LRFD Manual, chapter F)

Note: R_y is included to estimate the actual expected beam strength instead of just using the nominal plastic strength F_yZ. R_y has conservatively not been included in My

M_{nx} = 50(1.1)(936/12) O 1.5(50)(809/12)
= 4290 Kip-ft.< 5056 Kip-ft Okay

$P_u/\varphi P_n$ = 586/2654 = .221 > 0.2 Therefore, use LRFD eq. H1-1a

$P_u/\varphi P_n + (8/9)[M_{ux}/\varphi_b M_{nx} + M_{uy}/\varphi_b M_{ny}]$ O 1.0 (LRFD equation H1-1a)
586/[(.85)(2654)] + (8/9)[[3190/(.9)(4290)]+0] = .260 + .734

= .994 O 1.0 okay. Therefore, use <u>W36 x 232 beam</u>

If the axial load of 293 kips applied equally to both ends of the beam, and the top and bottom flange bracing is limited to the mid-span only, the same beam size is still adequate. Providing quarter point bracing along the beam span will increase the capacity of the beam.

B1 = 1/[1 - (293)/33,058)] = 1.009

M_{ux} = 1.009 (3133) = 3161 ft-kips

$\lambda = KL \sqrt{F_y/E} / (r_y)(\pi) = 0.908 \quad \lambda^2 = 0.825$

$\varphi P_n = (.85)(.658^{.825})(50)(68.1) = 2049$ kips

$P_u/\varphi P_n = 293/2049 = .143 < 0.2$ therefore use LRFD eq. H1-1b

$P_u/2\varphi P_n + M_{ux}/\varphi_b M_{nx} + M_{uy}/\varphi_b M_{ny}$ O 1.0 (LRFD equation H1-1b)
$293/(2)(.85)(2049) + 3161/(.9)(4290) + 0 = .084 + .819$

$= 0.903$ O 1.0 okay. Therefore, use W36 x 232 beam

As can be seen from above, the additional bracing points (at the quarter points) along the beam span allow the same W36x232 beam to be utilized even when using the conservative approach of applying all axial loads from one end of the beam.

Notes:

1. The ends of the W36 beam were assumed to be "pinned-pinned." This is a conservative assumption and leads to a large beam size (W36). In reality, the gusset plates at the end of the beam (for the braces that go up to the next level) provide fixity at the ends of the beams. Instead of calculating the effect of Q_b with $M = Q_bL/4$ (pinned ends), the moment "M" could be calculated as $M \approx Q_bL/8$ (with fixed ends), cutting the portion of the beam moment due to Q_b by 50 percent. See section 4.4, V and Inverted V (Chevron) Braces, for a discussion of beam-end fixity.

2. The selected W36x232 beam size is larger than the beam size used in the computer model and is a result of the unbalanced brace force on the beam. To be technically correct, the computer model should be re-run for the larger beam size. The revised brace-frame stiffness and resulting forces do not change the currently selected brace-frame member sizes.

B3.0 SCBF CONNECTION DESIGN

Need to determine:

B3.1 Brace weld to gusset plate
B3.2 Brace block shear
B3.3 Gusset plate size and compression check (lower end of brace)
B3.4 Gusset plate weld to beam and column
B3.5 Beam web to column flange connection

B3.6 Gusset plate weld to beam (upper end of brace)
B3.7 Gusset plate edge buckling

B3.1 Brace Weld to Gusset Plate

B3.1.1 Connection Design Forces

Gusset Tensile Check: Per 2002 AISC *Seismic Provisions, 1*3.3b, design tensile strength determined from the limit states of tension rupture and block shear rupture strength, shall be greater than or equal to the brace tensile force R_u:

$P_{ut} = R_y F_y A_g$ = (1.3)(46)(11.6) = 694 kips

Gusset Compression Check: Per 2002 AISC *Seismic Provisions,* 13.3c, the design compression strength, determined from the plate buckling limit state, shall be greater than the buckling strength of the brace of $F_{cr}A$. For the buckling strength of the gusset plate, the actual brace length should be used (approximately 13 feet) so as not to underestimate the brace compression capacity. Brace compression strength is as follows:

$\lambda_c = (13)(12)\sqrt{46/29000}/(2.63\,\pi) = .752$; $\lambda_c^2 = .5655$

$\varphi_c F_{cr}/F_y = .672$ (table 4, LRFD Manual, 3rd Ed., page 16.1-147)

$\varphi_c F_{cr} = .672(46) = 30.9$

$F_{cr} = 30.9/.85 = 36.4$ ksi

$P_{uc} = F_{cr}A = (36.4)(11.6) = 422$ kips < 694 kips

B3.1.2 Weld Size

Determine size and length of weld, tube brace to gusset plate. Check shear lag fracture at net section of brace.

Assume gusset plate one inch thick (Note: the final size is 7/8 inch thick; see following calculations).

Fillet Weld Strength:

Assume 9/16 inch fillet welds (four locations)

For fillet weld: $\varphi R_n = \varphi F_w A_w$, $\varphi = .75$, $F_w = .6 F_{exx}$

$F_w = .6(70) = 42$ ksi, E70xx electrodes

$\varphi R_n = .75(42)(.707)(9/16) = 12.5$ kips per inch of weld

Weld Length Required:

Length of 9/16 inch fillet weld required:

L_w= 694/[(4 welds)(12.5)] = 13.9″ where 694 kips = P_{ut}(demand)

Use 15″ of 9/16 fillet welds

B3.2 Brace Block Shear and Shear Lag Fracture

B3.2.1 Brace Wall Rupture at Weld

Check block shear capacity of tube brace member sidewalls along the four fillet weld lengths to the gusset plate.

HSS 7x7x1/2 wall thickness t = 0.465″

F_u = 58 ksi for grade B, F_y = 46 ksi

A_{nv} = (0.465)(4 sides)(15) = 27.90 in^2 ; A_{nt} = 0 in^2

The slot length cut in the HSS tube brace is typically longer than the fillet weld length from brace to gusset plate for erection purposes. Therefore, no connection exists between the brace slot width and the end of the gusset plate to develop block shear tension.

$\varphi R_n = \varphi(.6F_uA_{nv})$ = 0.75 (0.6)(58)(27.9) = 728 > 694 kips Okay

B3.2.2 Check Shear Lag Fracture at Net Section at Slot in Tube

$\varphi_t Rn = (\varphi_t)(F_u)(A_e)$ (From LRFD, eq. D1-2)

In the authors' opinion, there should be a "R_f" value corresponding to R_y. The actual F_u value would be higher than the specified F_u value. Not multiplying the right side of the equation by R_f seems overly conservative. This is not currently addressed in the AISC provisions. To comply with the current AISC provisions, R_f is <u>not</u> used in the calculations below.

The typical connection method of cutting a slot in the HSS brace member for the gusset plate will always result in the brace net section being inadequate to develop the brace's expected gross area tensile capacity ($A_gF_yR_y$). Reinforcement plates will always be required to increase the brace net section area, in order to address shear lag. The total required cross-sectional area of the reinforcement plates will approximately equal the HSS net cross-section area without the reinforcement plates. In other words, typically the total area of the reinforced net section area will be about twice the HSS net area section at the gusset plate slot by itself. This seems like a lot of reinforcement plate for the HSS brace for so little area being removed, and is due to the reduction in design capacity resulting from the shear lag coefficient *U*, the phi factor of fracture $\varphi_t = 0.75$ for tensile rupture, and having $R_yF_y \approx F_u$.

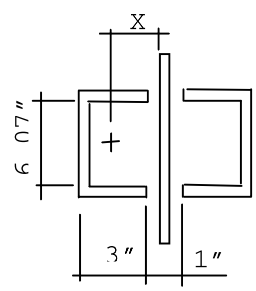
LRFD Manual, chapter J, section J5.2 (b) states that A_n shall not exceed $0.85A_g$. In the authors' opinion, this check should only be done after the reinforcement plates are included, since the total reinforced net section area (A_n')at the brace connection will now exceed the brace member's nominal gross area A_g. The brace member's gross area (A_g) will now be the brace member net area (A_n) for this check since the reinforced section has a greater cross-sectional area (A_n').

$A_e = UA_n$ (LRFD eq. B3-1), where $U = 1-(X/L_w)$

Where X is the distance to the centroid of the half section (at the net section) of the tube measured from the face of the gusset plate. Assuming a one inch thick gusset plate, X is calculated as shown here.

W1 = 7″ – 2(0.465) = 6.07″
A1 = 6.07 x 0.465 = 2.822 sq. inches
D1 = 3 – 0.465/2 = 2.77″

W2 = (7″-1.0″)/2 = 3.0″
A2 = 2x3x0.465 = 2.790 sq. inches
D2 = 3/2 = 1.5″



L_w = length of fillet weld to gusset plate = 15″

X = [(2.790 x 1.5) + (2.822 x 2.77)]/(2.790 + 2.822)
= 12/5.61 = 2.14″

Note: the actual slot width in the HSS brace will exceed the gusset plate thickness. This can be ignored since the shear lag is calculated to the face of the gusset plate and not the edge of the slot.

Alternately, see AISC HSS Connections Manual, section 2.1:
$X = (B^2 + 2BH)/(4(B+H))$ to gusset plate centerline.
For square HSS, this reduces to X = 3B/8) to gusset plate centerline. Assuming a one inch thick gusset plate:
X = 3B/8 – 1/2″ = ((3 x 7)/8)-.5 = 2.125 ≈ 2.14.

U= 1-(2.125/15)= 0.858

A_n = (11.6)-(2 x 1.0 x .47) = 10.7 sq. inches

A_e = .858(10.7) = 9.18

φR_n = (.75)(58)(9.18) = 399 < 694 kips No Good (N.G.)

Therefore, the ends of the tube brace at the net section (at the slot) need to be reinforced to increase the net area.

<u>Reinforcement Plates (F_y = 50 ksi):</u>

Approximate net area required ≈ 694/399 x 10.7 = 18.6 sq. inches

Since HSS and reinforcement plates have different yield (F_y) and tensile (F_u) strengths, increase the required reinforcement area by the ratio of [brace (F_y)/reinforcement plate (F_y)]. The reinforced brace net section will be ultimately checked based upon tensile strength (F_u) once the reinforcement plates have been sized.

Net area required =(18.6 – 10.7)(46)/(50) = 7.27 sq. inches

7.27 divided by 2 sides = 3.64 sq. inches per side

The reinforcing plate width can be either narrower or wider than the HSS section flat width, calculations are shown for both cases.

B.3.2.2.1 Reinforcing Plate <u>Narrower</u> than HSS Flat Width

HSS corner radius = 3t maximum.
Assume can fillet weld HSS flat width within 1.5t of vertical face (note that the fillet weld may not have equal leg lengths due to welding on the HSS corner radius.

HSS 7x7x1/2 Flat width = 7″-(3t) = 7″-(3)(0.5)= 5.5″max
Assume 1/2″ fillets for reinforcing plate to HSS:
Plate width = 5.5-(2)(1/2)= 4.5″

Try reinforcing plates 1″ x 4.5″ (F_y = 50 ksi steel):
Assume U = 0.80 to help size reinforcement plate.
As = 4.5 in^2 ≈ 3.64/0.8 = 4.55 in^2 okay).

Weld reinforcing plates to flat vertical sides of tube brace with 1/2″ fillet welds. Recalculate A_e since X and A_n have changed.

$U' = 1-(X'/l_w)$

Where "X'" is the revised distance to the centroid of the half section (at the net section) of the tube, including the reinforcement plate measured from the face of the gusset plate.

$X' = [12+(3.50 \times 1.0 \times 4.5)]/[5.61+(1.0 \times 4.5)] = 2.74''$

$U' = 1 - (2.74 /15) = 0.82$

$A_n' = 10.54 + (2 \times 1.0 \times 4.5) = 19.54 \text{ in}^2$

$A_g / A_n' = 11.6 /19.54 = 0.59 < 0.85$ okay per LRFD section J5.2

$A_c' = U'A_n' = .82(19.54) = 16.02 \text{ in}^2$

Since brace and reinforcing plate steel grades have different ultimate strengths, check reinforced net section capacity based on the lower HHS steel F_u value.

Conservatively:

$\varphi R_{n(HSS+Reinf.Plate)} = (.75)(58)(16.02) = 697$ kips > 694 Okay.

<u>Use 1" x 4.5" reinforcing plates</u> (see figure B-7).

Note: HSS F_yR_y exceeds HSS F_u which means the actual F_u minimum is greater than 58 ksi, but, as discussed by the authors in section B3.2.2, the current AISC *Seismic Provisions* do not allow R_f to be used. If the actual F_u (R_fF_u) for the HSS was 65 or greater, than a thinner reinforcing plate could have been used based upon the F_u of the reinforcing plate.

Length of reinforcing plates to develop strength of plates using one-half inch longitudinal fillet welds to HSS 7x7:

Since the reinforcement plates and HSS are of different F_u, develop the reinforcement plate based upon the lower F_u value (HSS F_u = 58 ksi < reinforcement plate F_u = 65 ksi).

Plate strength at fracture: (1 x 4.5 x 58 ksi) = 261.0 kips

$\varphi Rn = .75(.707)(1/2)(.6)(70 \text{ ksi}) = 11.14$ kips per inch
(one-half inch fillet welds, E70xx electrodes)

The reinforcement plate is typically welded all around especially if exposed to weather to prevent moisture intrusion between the reinforcement plate and HSS brace. Therefore, both transverse and longitudinal fillet welds are used to develop the reinforcement plate strength.

Length of weld required:

L = [261- (4.5″ x 11.4)]/(2 x 11.14) = 9.4″ say 10″

Length of HSS brace slot beyond end of gusset for erection:

Assume one half of brace width =(7/2) = 3.5″ for HSS 7x7

Reinforcement plate length = (2 x10″)+3.5″ = 1′- 11 1/2″ long.

<u>Use plates 1′-11½″ long centered on the 3.5″ over-slot</u>

B.3.2.2.2 Reinforcement Plate <u>Wider</u> than HSS Brace Nominal Width

Area required:

7.27 divided by 2 sides = 3.64 sq. in. per side

Plate width wider than HSS 7x7 = 7″ + 1/2" +1/2″ = 8″
Assume U = 0.80 to help size reinforcement plate.
Plate thickness = 3.64 /(0.8 x 8″) = 0.57″ $\Rightarrow$ 0.625 = 5/8″
B/t = 8 /(5/8) = 12.8 < (110/$\sqrt{F_y}$ = 110/$\sqrt{50}$= 15.55

<u>Use 8″x 5/8″ plate</u> = 5″ sq. in.

Calculate Shear Lag coefficient and Net Area:

X′ = [12+(3.313 x .625 x 8.0)]/[5.61+(.625 x 8.0)] = 2.69″

U′ = 1 – (2.69 /15) = 0.82

A_n′ = 10.54 + (2 x .625 x 8.0) = 20.54 in^2

A_g / A_n′= 11.6 /20.54 = 0.57 < 0.85 okay per LRFD section J5.2

A_e′ = U′A_n′ = (.82)(20.54) = 16.84 in^2

Conservatively:

$\varphi R_{n(HSS+Reinf.Plate)}$=(.75)(58)(16.84)= 733 kips > 694 Okay.

Welding to HSS Brace (E70xx Electrodes):

Plate strength at fracture:(0.625 x 8.0 x 58 ksi) = 290.0 kips
Flare Bevel Weld effective thickness = (5/16)R
HSS 7x7 Flat width = 5.6″(use 5.0″ for welding on flat width)
(Use one-half inch fillet for welding on flat width = 11.14 k/in, do not wrap weld around corner, per AWS D1.1)

R = HSS corner radius, use R = 2t = 2 x .465 = 0.93

φRn = .75(.6)(70 ksi)(5/16)(0.93) = 9.15 kips per inch

Length of weld required:

L = [290 - (5.0″ x 11.14)]/(2 x 9.15) = 12.8″ say 14″

Length of HSS brace slot beyond end of gusset for erection:
Assume (7/2)= 3.5″

Reinforcement plate length = (2 x14″)+3.5″ = 2′- 7 1/2″ long.

Use plates 2′-7½″ long centered on the 3.5″ over-slot

The authors recommend that the reinforcing plate be made wider than the HSS width as opposed to thicker and narrower in width. The reinforcement plate should comply with the same b/t ratio requirements as the brace. The flare grove weld could be reinforced per AISC/AWS with a reinforcing fillet to reduce the required weld length (reinforcing fillet calculation not shown).

Another alternate would be to use a 7/8 inch thick x 5 inch wide reinforcement plates with a partial penetration weld to the HSS instead of fillet welds. The fabricator will likely prefer the 1 inch x 4.5 inch plate which requires less welding to attach the reinforcement plates than using the flare bevel welds for the wider reinforcement plates to the HSS radii.

The actual gusset plate used is 7/8 inch thick, the shear lag coefficient "*U*" calculations could be redone for the 7/8 inch gusset thickness, but this will result in no design changes for the reinforcement plates. As can be seen from the design example, the length of the reinforcement plate will be in the range of four to five times the reinforcement plate width.

B3.3 Gusset Plate Size and Compression Check

Design gusset plate at lower end of brace.

B3.3.1 Check Compression in Plate at Whitmore Section

Assume 7/8 inch thick plate. (ASTM A572 steel, grade 50)

Whitmore width (see figure B-6):

W = D + 2(L_w)(Tan 30°) = 7 + (2)(15)(.577) = 24.3″

Check compression in gusset beyond end of brace:
K = 1.2 (assumed); $L_{g\ avg}$ = 18″ (average length 1″ wide strip)

([L_{g1} + L_{g2} + L_{g3}]/3 = $L_{g\ avg}$; see figure B-6A)

$r = .288t = .288(7/8) = .25$

$Kl_g/r = 1.2(18)/.25 = 86.4$; $\varphi F_{cr} = 24.6$ ksi (LRFD table 3-50)

$\varphi F_{cr}A = 24.6(7/8)(24.3) = 523$ kips > Brace $P_{uc} = 422$ kips Okay

B3.3.2 Check Tension in Plate at Whitmore section

$\varphi F_yA = .9(50)(24.3)(7/8) = 957$ kips > $P_t = 694$ kips Okay

B3.3.3 Check Block Shear Rupture in Gusset Plate (LRFD Section J4)

$P_u = 694$ kips (tension capacity of brace)

$F_u = 65$ ksi for grade 50; $A_{nv} = (.875)(2)(15) = 23.6\ in^2$

$A_{nt} = (.875)(7) = 5.5\ in^2$; $F_uA_{nt} = (65)(5.5) = 357$ kips

$.6F_uA_{nv} = (.6)(65)(23.6) = 920$ kips, therefore $F_uA_{nt} < .6\ F_uA_{nv}$

$\varphi R_n = \varphi(.6F_uA_{nv} + F_yA_gt)$ O $\varphi(.6F_uA_{nv} + F_uA_nt)$
$\varphi R_n = .75[920 + (50)(5.5)] = 896 < .75(920 + 357) = 957$ kips

$P_u = 694$ kips $< \varphi R_n = 896$ kips, okay.

Therefore, use 7/8 inch gusset plate, A572 Gr. 50 Steel

B3.4 Gusset Plate to Beam and Column Connections

Gusset plate at lower end of braces (columns):

See figure B-5 for free-body diagram showing brace axial force resolved into corresponding moments, vertical and horizontal forces on gusset plate using the Uniform Force Method (see AISC LRFD Manual). Since the gusset plate geometry is set by column and beam depths, the brace angle "θ", and the 2t dimension, the gusset plate welds are handled like an existing connection since the idealized "α" and "β" dimensions cannot be achieved. In the LRFD Manual, a method is shown to handle this condition. The actual centroids at the gusset-to-column and gusset-to-beam interfaces are given by α′ and β′. Setting β = β′, no moment would exist at the gusset-to-column interface. Because α does not equal α′, a moment M_b exists on the gusset-to-beam connection, where $M_{ub} = V_{ub}(\alpha - \alpha')$. See the LRFD Manual (Uniform Force Method) for a definition of the terms used in the following calculations.

B3.4.1 Gusset Plate Forces and Dimensions

$\alpha = 22.5''$; $\alpha' = 17''$; $\beta = \beta' = 9''$; $e_b = 18.17''$; $e_c = 7.31''$

$r = [(\alpha + e_c)^2 + (\beta + e_b)^2]^{.5} = 40.3''$

Note: e_b is based upon the computer model W36x182 beam size. The revised numbers would be slightly different for the selected W36x232 beam. But since the difference is so small (18.17″ vs. 18.56″), the calculations have not been revised.

Brace Tension Capacity:

$P_{max} = P_{ut} = R_yF_yA_g = 694$ kips

$V_{uc} = \beta P_u/r = 155$ kips; $H_{uc} = e_cP_u/r = 125.9$ kips

$V_{ub} = e_bP_u/r = 312.9$ kips; $H_{ub} = \alpha P_u/r = 387.5$ kips

$M_{ub} = V_{ub}(\alpha - \alpha') = 1721$ inch-kips

Brace Compression Capacity:

$P_{max} = P_{uc} = 422$ kips

$V_{uc} = \beta P_u/r = 94$ kips; $H_{uc} = e_cP_u/r = 76.3$ kips

$V_{ub} = e_bP_u/r = 189.8$ kips; $H_{ub} = \alpha P_u/r = 235.0$ kips

$M_{ub} = V_{ub}(\alpha - \alpha') = 1044$ inch-kips

B3.4.2 Check Gusset Plate and Size Gusset Plate-to-Beam Weld

Welded gusset plate length "d" = (2′-10″)- 1/2″ = 33.5″
Use effective weld length d = 33″ (see figures B-5 and B-7)

$S_{pl} = bd^2/6 = (7/8)(33)^2/6 = 159\ in^3$

Weld Stresses (governed by brace tension):

Bending: $f_{bb} = M/S = 1721/159 = 10.8$ ksi

Shear: $f_{vb} = H_{ub}/A_{pl} = 387.5/(33\ x .875) = 13.4$ ksi

Tension: $f_{tb} = V_{ub}/A_{pl} = 312.9/(33\ x\ .875) = 10.8$ ksi

Weld Peak Stress: $f_{rb} = \sqrt{(10.8+10.8)^2+13.4^2} = 25.4$ ksi

From LRFD Manual 3rd Ed., page 13-11, "Design Strength": the weld should be designed for the larger of the peak stress or 1.4 times the average stress:

$f_{avg.} = (1/2)\left[\sqrt{(10.8-10.8)^2+13.4^2}+\sqrt{(10.8+10.8)^2+13.4^2}\right] = 19.4$ ksi

$1.4 \times 19.4 = 27.2$ ksi > peak stress = 25.4 ksi
Average stress governs weld design

Try 5/8″ fillet welds each side:

$F_u = .6(70) = 42$ ksi (E70xx electrodes)

$\varphi R_n = .75(42)(.707)(5/8)(2 \text{ sides}) = 27.8$ k/in.

27.8 k/in > (27.2)(7/8) = 23.8 k/in. Okay

<u>Use 5/8 fillet welds or complete penetration weld at gusset plate to beam.</u>

Note: In the above calculation to determine f_{avg}, the average bending stress f_{bb},one-half of the weld bending stress is in tension and one-half is in compression; therefore, the acting stresses are calculated for each half length of the gusset plate and the weld stress is taken as the average of both half lengths for comparison with the peak weld stress. Note, there is a current discussion about reducing the 1.4 factor to 1.25, but this has not as yet been officially adopted by AISC.

Gusset plate shear strength check (plate F_u = 65 ksi):

$\varphi R_n = \varphi(.6F_uA_{nv}) = 0.75\ (0.6)(65)(1″\text{x}0.875″)$
$= 25.6$ k/in > 23.8 k/in Okay

B3.4.3 Check Beam Web Crippling (LRFD K1.4)

Beam size W36x232: $t_f = 1.52″$, $t_w = .87″$, $d = 37.12″$

Web crippling okay by inspection since gusset plate is the same thickness as the beam web but calculation is shown for completeness.

N = Gusset plate length = (2′-10″)- 1/2" = 33.5″ use 33″
N/d = 33/37.12 = 0.89 > 0.2
AISC equation (K1-5b): $\phi = 0.75$

$$0.4(t_w)^2[1 + (((4N/d)-0.2)(\ t_w/t_f)^{1.5})]\sqrt{EF_{yw}t_f/t_w}\ )$$

$$((4N/d)-0.2) = [(4 \times 33)/37.12] - 0.2 = 3.36$$

$$(t_w/t_f)^{1.5} = (0.87/1.52)^{1.5} = 0.433$$

$$\sqrt{EF_{yw}t_f/t_w} = \sqrt{(29000)(50)(1.52/0.87)} = 1591.6$$

$0.4(0.87)^2[1+(3.36 \times 0.433)](1591.6) = 1183$ kips

$(0.75)(1183) = 887$ kips

887 kips > V_{ub}(compression brace) = 189.8 kips Okay

Note: If beam web is thinner than the gusset plate, web crippling should always be checked per LRFD, section K1.4.

Web Yielding:

Beam size W36x232: k = 2.5”, t_w = .87”,

(AISC K1.3) $R_n = (2.5k + N)\ F_{yw}\ t_w,\ \phi = 1.0$
$= ((2.5 \times 2.5) + 33)(50 \times 0.87) = 1707$ kips

, $\phi\ R_n$ = 1707 kips > V_{ub}(compression brace) = 189.8 kips Okay

B3.5 Beam Web Connection to Column Flange

Beam at third floor, lines two or four. Calculations shown below are for connection at line four.

B3.5.1 Dimensions and Forces

Use 30 inches for height of beam web shear plate = l_{bw} (refer to figure B-5).

The beam web-column connection needs to be evaluated for four different possible loading conditions as a result of the braces below the beam mid-span having either yielded or remaining un-yielded. The connection design includes both beam shear and beam axial loads.

Determine maximum shear and axial forces on beam connection:

<u>Brace above at end of beam in tension (yielded)</u>:

V_{bu} = vert. component (tension brace above beam)= 312.9 kips

Case 1: R_{bu} = reaction from buckled brace below beam
= $V_{max} = (WL/2) + Q_b/2 = 218$ kips

Case 2: R_{bu} = reaction from brace not buckled below beam
= $V_{max} = (WL/2) = 17.7$ kips

$R_{bu} - V_{bu} = V_{max} - V_{bu} = 218 - 312.9 = -94.9$ kips (uplift)

$R_{bu} - V_{bu} = V_{max} - V_{bu} = 17.7 - 312.9 = -295.2$ kips (uplift)

Axial = $H_u - H_{ub} = H_{uc} = e_c P_u / r = 125.9$ kips

Resultant force: = $\sqrt{295.2^2 + 125.9^2}$ = 320.9 kips

<u>Brace above at end of beam in maximum compression (P_{uc})</u>:

V_{bu} = vert.component (compression brace above beam)=189.8 kips

Case 3: R_{bu} = reaction from buckled brace below beam
= V_{max} = (WL/2)+ Q_b/2 = 218 kips

Case 4: R_{bu} = reaction from brace not buckled below beam
= V_{max} = (WL/2) = 17.7 kips

R_{bu} + V_{bu} = 218 + 189.8 = +407.8 kips

Axial = H_u - H_{ub} = H_{uc} = e_cP_u/r = 76.3

Resultant force: = $\sqrt{407.8^2 + 76.3^2}$ = 414.9 kips > 320.9 kips

Note that only the horizontal force H_{uc} needs to be transferred through the beam-column connection since the beam force H_{ub} is transferred directly from the gusset plate to the beam.

<u>Diaphragm Drag force:</u>

Drag force from beam at level four between lines 4 & 5, 1 & 2:

From computer run results, drag force = 28 kips

From hand calculation (level four diaphragm):

Drag force = (1.25'+30')(77.5'/2)(27.7 psf)= 33.5 kips

Factor this drag force up by a ratio of axial force in beam (generated from the greater of the following two load cases show below) to the axial load in beam from computer run results:

Case 1: Buckled Brace below the beam:
Load case 1.2D+.5L+Qb: Beam P_u = 293 kips

Case 2: Unbuckled Braces below the beam:
Beam P_u =(Brace P_{uc} max)(15.0/20.18)= 422(0.743)= 314 kips

The axial load in the beam at level four from the computer run results is 162 kips.

Factoring the drag force, assuming equal drag force from each end of frame since this is a symmetrically located two-bay frame:

H_{ub} = (314/162)(33.5) = 1.94(33.5) = 65 kips

Check collector force including omega: 2.2 x 28 = 61.6 kips

B3.5.2 Beam Web Connection to Column Flange

Forces:

Maximum Compression in brace above beam governs connection design since the connection resultant force is larger.

Design beam web connection for V_{ub} = 407.8 kips
H_{uc} = 76.3 + 65 = 141.3 kips

f_v = 407.8 kips/30″ = 13.6 k/in.

f_a = 141.3/30 = 4.7 k/in.

$f_r = \sqrt{13.6^2 + 4.7^2}$ = 14.4 k/in.

Multiply by 1.4 (stress distribution factor):

f_r = 1.4(14.4) = 20.2 k/in.

Weld Design:

Try fillet weld 5/8″ (One-sided fillet weld with E70 electrodes):

φR_n = .75(42)(.707)(5/8) = 13.9 < 20.2 N.G.

Try complete penetration (CP) weld (beam web to column flange):

Check beam web shear:(W36x232; t_w=0.87″, F_y=50ksi, F_u=65 ksi)
Check beam web CP weld:

Weld: φR_n = .75(42)(.87)= 27.4 k/in. > 20.2 k/in okay

Beam Web: $\varphi R_n = \varphi(.6F_uA_{nv})$ = 0.75 (0.6)(65)(0.87)
= 25.4 k/in > 20.2 okay

Therefore use complete penetration weld at beam web to column flange, using the shear plate as a weld backup plate. The shear plate also acts as erection connection, but does not have to transfer the beam web connection force to the column flange, since there will be a direct connection of the beam web to the column flange via the CP weld.

Use beam web shear plate ½ x 4″x 30″. Weld plate to column flange with ¼″ weld both sides. See figure B-7.

B3.5.3 Check Gusset Plate and Size Gusset Plate-to-Column Weld

Height gusset plate = 1′-6″ = 18″ Effective weld length = 17″

Brace Tension Forces Govern Design:

Gusset Shear: f_{vc} = 155/(17x.875) = 10.4 ksi

Gusset Tension: f_{hc} = 125.9/(17x.875)= 8.5 ksi

$f_{rc} = f_{avg} = \sqrt{10.4^2 + 8.5^2}$ = 13.4 ksi; 1.4 x 13.4 = 18.8 ksi

(Since there is no moment at the gusset plate-column interface per the Uniform Force Method (β = β′), the resultant peak stress "fr" is also equal to the average stress)

Try 3/8″ fillet weld each side:

Gusset shear: 18.8 x .875 x 1 = 16.4 k/in.

φR_n = .75(42)(.707)(3/8)(2) = 16.7 k/in. > 16.4 Okay.

Use 3/8 fillet welds at column

B3.5.4 Check Column Web Crippling (LRFD K1.4) and Column Web Yielding (LRFD K1.3)

Instead of checking the column web for crippling and yielding, a horizontal stiffener has been added at the column web to stiffen the column flanges since a horizontal stiffener was provided to the gusset plate free edge (see figure B-7). If a horizontal stiffener wasn't added to the gusset plate, then the column web would be checked per K1.3 and K1.4 to determine if a horizontal stiffener is required.

B3.6 Gusset Plate at Upper Ends of Braces (at Beam)

Check two loading conditions (refer to figure B-8):

1. Each brace has +/-P_{uc} axial loads (just before the compression brace buckles): horizontal forces are split 50-50 between tension and compression braces; shear and bending act on gusset plate, vertical axial = 0

2. Check when compression brace buckles and tension brace yields: shear, bending and vertical axial loads act on gusset plate.

Condition 1 - Brace forces equal:

P_{uc} = +/-$F_{cr}A$ = (36.4)(11.6) = +/-422 kips

H_u = (422 + 422)(15/20.18) = 627 kips

W36x232 d/2 = 18.56″

M_u = 627 x 18.56 = 11,637 in-kips

P_u = 0 (brace vertical components cancel each other)

Condition 2-Tension brace yielded, compression brace buckled:

P_{ut} = 694 kips; P_{uc} = .3$F_{cr}A$ =(.3)(36.4)(11.6) = 126 kips

H_u = (694 + 126)(15/20.18) = 609 kips

M_u = 609(d/2) = 609 x 18.56 = 11,303 inch-kips

P_u = (694 – 126)(13.5/20.18)= 380 kips

Properties and stress check:

Try 7/8 inch gusset plate:

$S_{pl} = bd^2/6$ = $(.875)(84^2)/6$ = 1029 in^3

Gusset Plate Stresses:

Condition 1:

Bending: f_b = M/S = 11,637/1029 = 11.3 ksi

Shear: f_v = 627/(84 x .875) = 8.5 ksi

Vert. Axial f_a = 0

$f_{avg.}$ = (1/2) $\left[\sqrt{(0-11.3)^2+8.5^2}+\sqrt{(0+11.3)^2+8.5^2}\right]$ = 14.2 ksi

1.4 x 14.2 = 19.9 ksi (controls)

Peak Stress = f_r = $\sqrt{11.3^2+8.5^2}$ = 14.2 ksi

Condition 2:

Bending: f_b = M/S = 11,303/1029 = 11.0 ksi

Shear: f_v = 609/(84 x .875) = 8.3 ksi

Vert. Axial fa = 380/(84 x .875) = 5.2 ksi

$f_{avg.} = (1/2)\left\lfloor\sqrt{(5.2-11.0)^2+8.3^2}+\sqrt{(5.2+11.0)^2+8.3^2}\right\rfloor = 14.2$ ksi

1.4 x 14.2 = 19.8 ksi

Peak Stress $f_r = \sqrt{(11.0+5.2)^2+8.3^2} = 18.2$ ksi

<u>Welding design:</u>

Try 3/8 fillet welds each side of gusset plate

φR_n = .75(42)(.707)(3/8)(2) = 16.7 k/in. (weld capacity)

Gusset plate shear strength check:

$\varphi R_n = \varphi(.6F_uA_{nv})$ = 0.75 (0.6)(65)(1x0.875) = 25.6 k/in. Okay

Gusset plate weld design force:

Gusset plate Shear = 19.9 ksi x.875 x 1 = 17.4 k/in.

φR_n = 16.7 k/in. < 17.4 k/in
(17.4/16.7 = 1.042 = 4.2% overstress say Okay.

The weld is only four percent overstressed by design and is considered to be acceptable. Less than five percent overstress in framing member and connection design is considered an acceptable standard of practice by many engineers. Note that Building department approval is required for the acceptance of any overstressed member or connection design. If the overstress is not acceptable, then the weld size would be increased to 7/16 inch fillet welds each side. Note that if the new proposed weld redistribution factor 1.25 is used instead of 1.4, than the weld peak stress of 18.2 ksi in condition 2 controls and 3/8 inch fillets are okay.

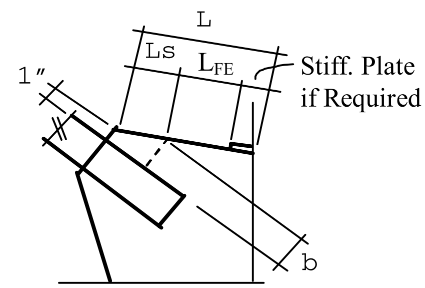
<u>Use 3/8 inch fillet welds at gusset plate-to-beam connection</u>

B3.7 Check Gusset Plate Edge Buckling

L = Length of gusset plate free edge

t = Gusset plate thickness, F_y = 50 ksi

The brace can be considered as stiffening the free edge of the gusset plate except when the perpendicular distance from the brace member surface to the free edge of the gusset plate exceeds the distance b as determined from the compact criteria equation: $b/t = 52 / \sqrt{F_y}$.



Distance b = $[(52)/\sqrt{50}](0.875) = 6.43''$
Length of stiffened free edge (Ls):
Ls = (6.43″-1″)/sin 30 = 10.86″

Free edge limit (L_{FE}):
$L_{FE}/t < .75\sqrt{E/F_y} = .75\sqrt{29000/50} = 18.1''$

Brace lower end (see figure B-6):
L (above brace) = 34.7″; L (below brace) = 19.34″
L (effective length) = L - Ls = 34.7-10.86 = 23.84″
L (effective length) = L - Ls = 19.34-10.86 = 8.48″
Below Brace: L_{FE}/t = 8.48/0.875 = 9.7″ < 18.1″
(No Stiffener required)
Above brace: L_{FE}/t = 23.84/.875 = 27.24″ > 18.1″
(Minimum Stiffener length = 27.24 - 18.1 = 9.14″)

Brace upper end (see figure B-8):
L (above brace)= 19.34″
L (below brace)= 19.34 + 46.08/2) = 42.38″
L (effective length)= L - Ls = 19.34 - 10.86 = 8.48″
L (effective length)= L - Ls = 42.38 - 10.86 = 31.52″
Above brace: L_{FE}/t = 8.48/0.875 = 9.7″ < 18.1″
(No stiffener required)
Below brace: L_{FE}/t = 31.52/.875 = 36.02″ > 18.1″
(Minimum stiffener length = 36.02 - 18.1 = 17.92″)

Brace Lower End (see figure B-7):

Top edge (above brace): Provide 1/2 " thick x 4″ wide stiffener plate along top edge of gusset plate. Stop stiffener plate 1″ from yield line (length stiffener plate ≈ 14″ long).

Lower edge (below brace): No edge stiffener plate required.

Brace Upper End (See Figure B-8):

Top edge (above brace): No edge stiffener plate required.

Lower edge (below brace): The length of the free edge is measured along the sloping edge plus half of the horizontal edge since the bracing is symmetrical. Along the sloping edge of the gusset plate no stiffener plate is required since the length is the same as above the brace. At the yield line, the gusset plate has been shaped so the free edge is horizontal (46.08″ long), and this edge needs to be braced due the length of the free edge and specifically because of the change in direction of the free edge. The horizontal stiffener plate should stop one inch from the yield line (stiffener plate length = (46.08/2)-1 = 22.04″ > 17.92″ minimum length from gusset mid-span). A vertical stiffener plate is added both sides at the center of the gusset plate.

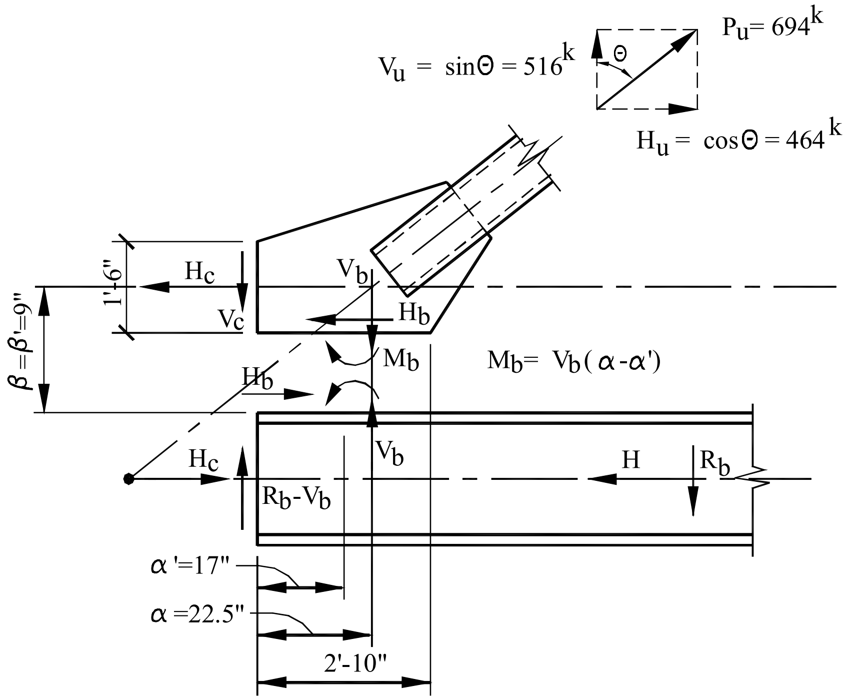
Other details and calculations for this brace to upper gusset plate connection are the same as at the lower end connection.

Beam Mid-Span Gusset Plate Notes:

1. An alternate beam mid-span detail is possible: The gusset plate at the beam for this detail (as shown in figure B-8) is about eight feet long with the work-point at intersection of the beam and brace centerlines. No moments exist in the beam due to eccentricity, since there is no eccentricity. If the work-point is moved down so that there is some eccentricity of the connection relative to the beam centerline, the gusset plate length could be reduced in length and possibly one of the stiffener plates eliminated (the plate free edge length would be reduced). The W36 beam would be able to handle the eccentric bending moments.

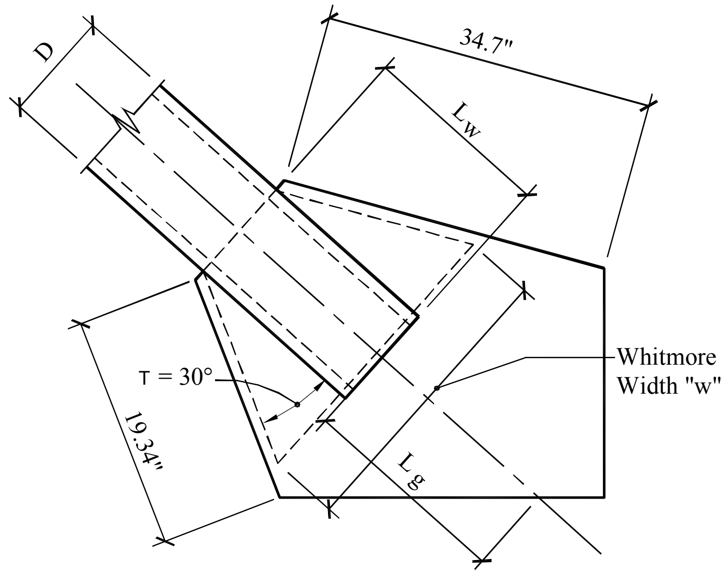
2. If a two-story X brace is used, then the bracing work-point should be located at the centerline of the beam.

B4.0 CONNECTION DETAILS



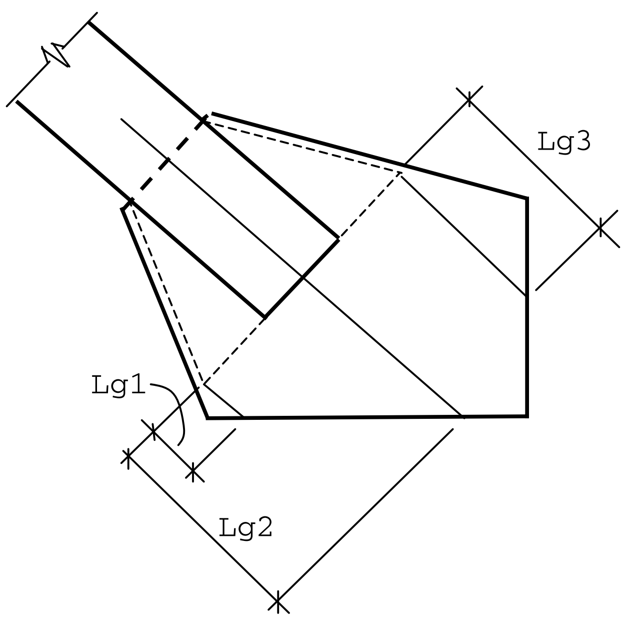
Uniform Force Method

Figure B-5



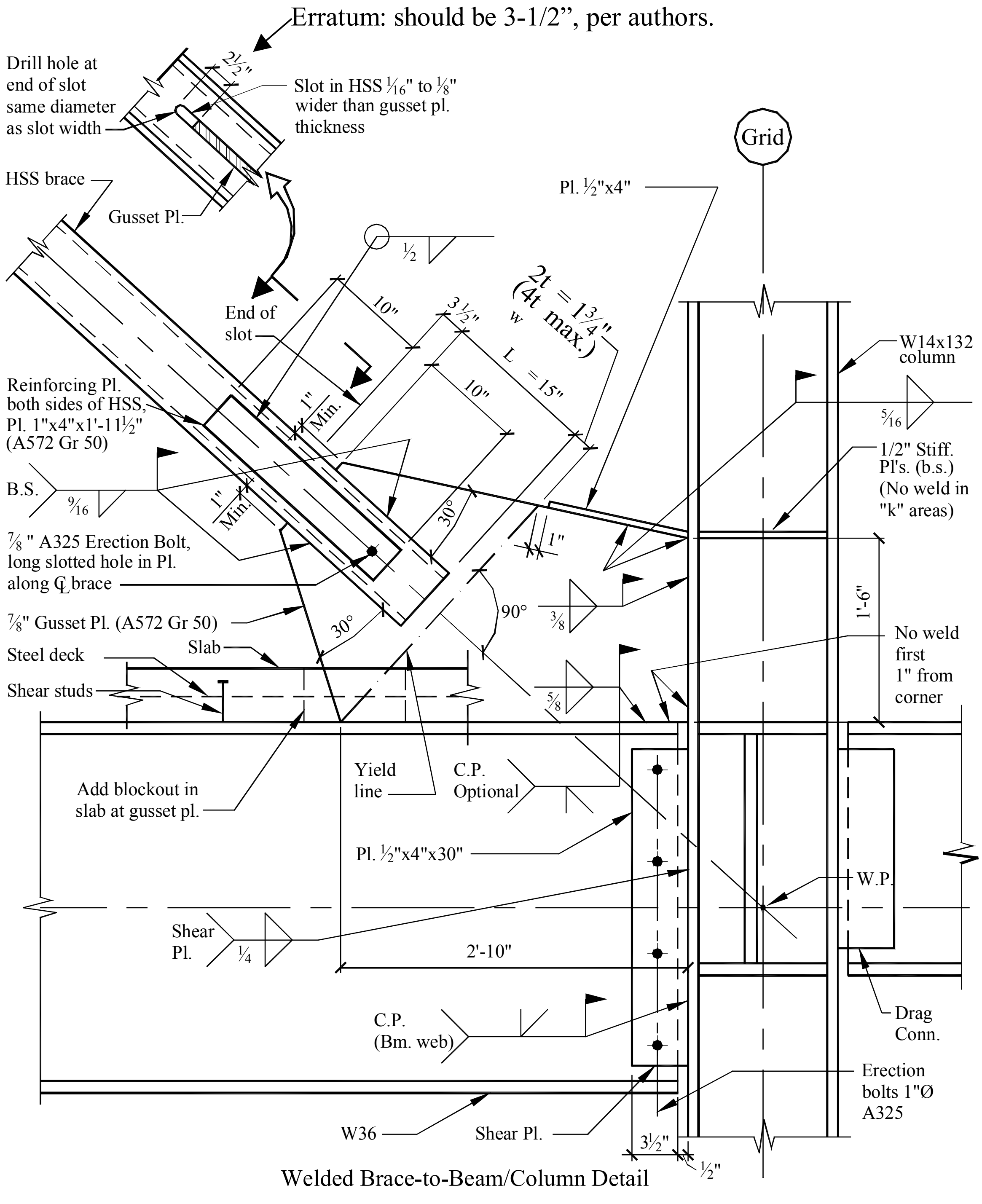
Gusset-to-Beam and Column Connection

Figure B-6



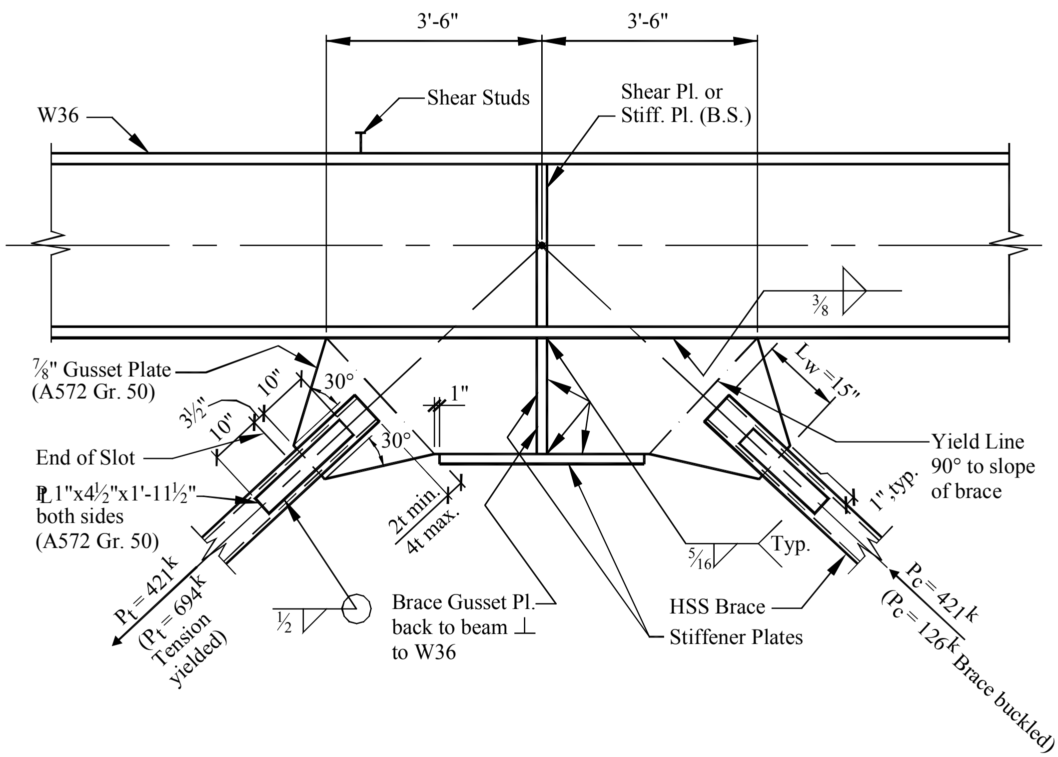
Gusset Plate Average Length Determination for Buckling

Figure B-6A



Welded Brace-to-Beam/Column Detail

Figure B-7



Upper Gusset Plate Proportions

Figure B-8

Note: If the gusset plate extends vertically below the beam bottom flange a significant distance, the authors recommend that an additional out-of-plane knee brace from the gusset plate back to a beam perpendicular to the gusset plate be provided to help brace the bottom of the gusset plate. The authors recommend defining the significant distance as being greater than the brace frame beam depth or 24 inches, whichever is less. This is not a building code requirement and is left up to the design engineer's discretion. An example of the knee brace is shown in the SEAOC *Seismic Design Manual*, Vol. III (updated for the 2000 IBC).

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