Steel TIPS
STRUCTURAL STEEL EDUCATIONAL COUNCIL
TECHNICAL INFORMATION & PRODUCT SERVICE

July 2004

Seismic Design of Buckling-Restrained Braced Frames

By

Walterio A. López
Associate
Rutherford & Chekene

And

Rafael Sabelli
Senior Associate
DASSE Design, Inc.

ABSTRACT

Buckling-Restrained Braced Frames (BRBFs) are a new steel seismic-load-resisting system that has found use in the western United States because of its efficiency and its promise of seismic performance far superior to that of conventional braced frames. The system is not yet addressed in the 2005 edition of the *AISC Seismic Provisions for Structural Steel Buildings*, but nevertheless a set of design provisions has been developed by AISC in conjunction with the Structural Engineer's Association of California. This report illustrates the seismic design of buckling-restrained braced frames; they are defined, and the provisions governing their design and required testing are explained. A summary of selected Buckling-Restrained Brace (BRB) testing performed to date is provided. Compliance with design requirements is explained through detailed component design of two typical BRBF configurations and development of testing protocols. A discussion of gusset-plate design and its influence on acceptable frame behavior is provided.

July 2004

Walterio A. López, Associate, Rutherford & Chekene, 427 Thirteenth Street, Oakland, CA 94612, wlopez@ruthchek.com

Rafael Sabelli, Senior Associate, DASSE Design, Inc., 33 New Montgomery Street, Suite 850, San Francisco, CA 94105, Sabelli@dasse.com

ACKNOWLEDGMENTS

The publication of this report was made possible in part by the support of the Structural Steel Educational Council (SSEC). The authors wish to thank Yuki Nakayama of the Kajima Corporation for his assistance in developing the Mathcad examples and many of the figures of this report. The authors wish to thank all SSEC members for their valuable comments. Particularly, special thanks are due to Abolhassan Astaneh-Asl, Michael Cochran, Patrick Hassett, William Honeck, Brett Manning, and Heath Mitchell for their valuable and detailed review comments. The authors also wish to thank Messrs. Ian Aiken of Seismic Isolation Engineering (SIE), Andy Hinchman of CoreBrace, and Steven Powell of Star Seismic for their valuable and detailed review comments.

The opinions expressed in this report are solely those of the authors and do not necessarily reflect the views of Rutherford & Chekene and DASSE Design where the authors are employed nor the Structural Steel Educational Council or other agencies and individuals whose names appear in this document.

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A_g	Gross area, in^2
A_{sc}	Area of the yielding segment of steel core, in^2
A_w	Web area, in^2
A_x	The floor area in ft^2 of the diaphragm level immediately above the story
C_b	Bending coefficient dependent upon moment gradient
C_d	Deflection amplification factor
C_{MAX}	Maximum compression in braces, kips
C_s	Seismic response coefficient
C_t (C_r)	Approximate period parameter
C_u	Coefficient for upper limit on calculated period
C_{vx}	Vertical distribution factor
DCR	Demand-capacity ratio
E	Modulus of elasticity of steel
F_a	Acceleration-based site coefficient
F_{cr}	Critical stress, ksi
F_i	Portion of the seismic base shear, V, induced at Level i
F_v	Velocity-based site coefficient
F_x	The design lateral force applied at story x
F_y	Yield stress, ksi
F_{yg}	Yield stress of gusset plates, ksi
F_{ysc}	Yield stress of steel core, kips
I	Occupancy importance factor
k	An exponent related to the structure period
L	Span length, in
L'	Clear beam distance, in
L_b	Laterally unbraced length, ft
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case (C_b > 1.0), ft
L_r	Limiting laterally unbraced length for inelastic lateral-torsional buckling, ft
L_{ysc}	Yielding length of the steel core, ft
M_n	Nominal flexural strength, kip-ft
M_p	Nominal plastic flexural strength, kip-ft
M_{pa}	Nominal plastic flexural strength modified by axial load, kip-ft
M_r	Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-ft
M_u	Required flexural strength, kip-ft

M_y	Moment corresponding to onset of yielding at the extreme fiber from an elastic stress distribution
P_{bx}	Axial load in a brace corresponding to the elastic story drift, kips
P_n	Nominal axial strength (tension or compression), kips
P_u	Required axial strength (tension or compression), kips
P_{ysc}	Yield strength of steel core, kips
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, kips
R	Response modification factor
R_y	Ratio of the expected yield strength to the minimum specified yield strength F_y
S_a	Spectral response acceleration
S_{DS}	Design earthquake spectral response acceleration at short periods
S_{D1}	Design earthquake spectral response acceleration at a period of 1 sec
S_{MS}	The maximum considered earthquake spectral response acceleration for short periods adjusted for site class effects
S_{M1}	The maximum considered earthquake spectral response acceleration at a period of 1 sec adjusted for site class effects
S_s	The mapped maximum considered earthquake spectral response acceleration at short periods
S_1	The mapped maximum considered earthquake spectral response acceleration at a period of 1 sec
S_x	Elastic section modulus about major axis, in^3
T	Fundamental period of the structure, sec
T_a	Approximate fundamental period of the structure, sec
T_{MAX}	Maximum tension in braces, kips
V	Base shear, kips
V_c	Story shear resisted by column, kips
V_n	Nominal shear strength, kips
V_p	Nominal shear strength corresponding to M_p, kips
V_{pa}	Nominal shear strength corresponding to M_p modified by axial load, kips
V_u	Required shear strength, kips
V_x	Story shear
W	Seismic weight
Z_x	Plastic section modulus, in^3
b_f	Flange width of rolled beam or plate girder, in
d	Overall depth of member, in
e_X	Eccentricity, ft
e_Y	Eccentricity, ft
f'_c	Specified compressive strength of concrete, psi
h	Clear distance between flanges less the fillet or corner radius for rolled shapes, in
h_n	The height above the base to the top of the building, ft
h_{sx}	Story height below level x, in

h_x	The height above the base level x, feet
k_x	Effective length factor about x axes for prismatic member
k_y	Effective length factor about y axes for prismatic member
l_x	Laterally unbraced length about x axes, ft
l_y	Laterally unbraced length about y axes, ft
r_{max}	The ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear
r_x	Radius of gyration about x axes, in
r_y	Radius of gyration about y axes, in
t_f	Flange thickness, in
t_w	Web thickness, in
w_x	Portion of seismic weight, W, that is located at or assigned to level x, kips
x	Approximate period parameter
β	Compression strength adjustment factor
Δ_a	Allowable story drift, in
Δ_b	Total brace axial deformation for the brace test or total brace end rotation for the subassemblage test, in or rad
Δ_{bx}	Brace axial deformation corresponding to the elastic story drift, in
Δ_M	Design story drift, in
Δ_x	Elastic story drift, in
Δ_{bm}	Brace axial deformation corresponding to the design story drift, in
Δ_{by}	Brace axial deformation at first significant yield, in
ε_{BRC}	Average brace strain
ϕ	Resistance factor
ϕ_b	Resistance factor for flexure
ϕ_c	Resistance factor for compression
ϕ_v	Resistance factor for shear
λ_c	Column slenderness parameter
λ_{ps}	Limiting slenderness parameter for compact element
μ	Brace ductility demand
ψ	Brace angle with respect to a vertical axis
θ_a	Allowable interstory drift ratio
θ_M	Design interstory drift ratio
θ_x	Elastic interstory drift ratio
ρ	A coefficient based on the extent of structural redundance present in a building
$\Sigma\mu_p$	Brace cumulative plastic ductility
ω	Tension strength adjustment factor

1. INTRODUCTION

Buckling-Restrained Braced Frames (BRBFs) are a relatively new type of concentrically braced system characterized by the use of braces that yield inelastically both in tension and compression at their adjusted strengths (Clark et al., 1999). Despite their being a relatively new system, BRBFs in the United States have to date been subjected to numerous analytical and experimental studies that have demonstrated their robustness when subjected to code-type ground motions (Clark et al., 1999; Fahnestock et al., 2003, López et al., 2002; Sabelli, 2001; Sabelli et al., 2003; and Uang and Kiggins, 2003). The brace component of BRBFs is known as the Buckling Restrained Brace (BRB).

BRBs have full, balanced hysteresis loops as illustrated in Figure 1, with compression-yielding similar to tension-yielding behavior. They achieve this through the decoupling of the stress-resisting and flexural-buckling resisting aspects of compression strength. Axial stresses are resisted by a shaped steel core. Buckling resistance is provided to that core by a casing, which may be of steel, concrete, composite, or other construction. Because the steel core is restrained from buckling, it develops almost uniform axial strains. Plastic hinges associated with buckling do not form in properly designed and detailed BRBs.

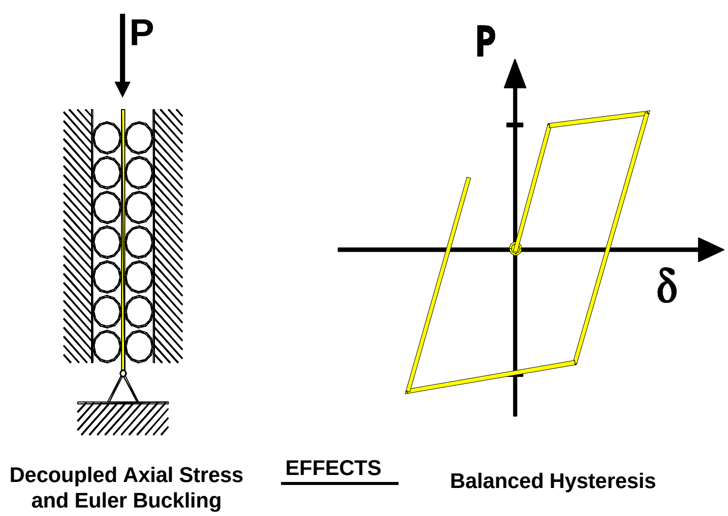


Figure 1. Mechanics of a Buckling-Restrained Brace

Figure 2 shows a schematic of a commonly used BRB. The steel core is divided into five segments: the restrained yielding segment, a reduced section within the zone of lateral restraint provided by the casing; restrained, nonyielding transition segments of larger area than the yielding segment; and unrestrained, nonyielding connection segments that extend past the casing and connect to the frame, typically by means of gusset plates.

By confining the inelastic behavior to axial yielding of the steel core, great ductility can be achieved by the brace itself. The ductility of the steel material is realized over the majority of the brace length. Thus the hysteretic performance of these braces is similar to that of the steel core material. The schematic hysteresis diagram in Figure 1 shows stable behavior and significant energy dissipation. Braces with steel cores that have significant strain-hardening will exhibit that behavior as well. A real hysteresis diagram also shows compression overstrength (a greater strength in compression than in tension). Some of this is attributable to the material behavior and some to a small transfer of stress to the casing.

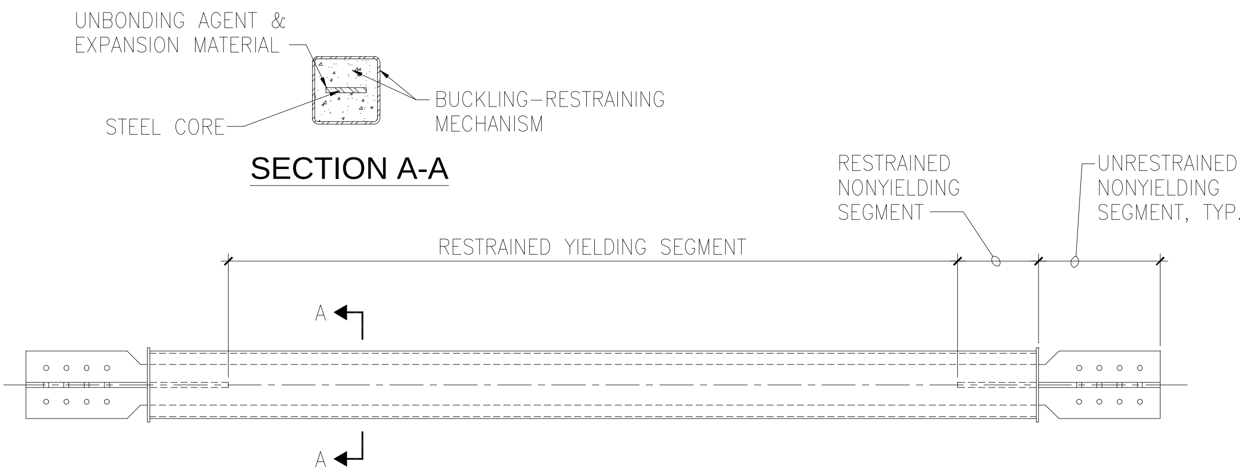


Figure 2. Illustration of a BRB Element (Adapted from Wada et al., 1998)

Several BRB concepts have been developed by researchers and manufacturers. BRB concepts vary in their use of single or multiple cores, their use of single or multiple-joined casings, the type of steel core, the core orientation, the expansion material, and the methods of preventing stress transfer to the casing. Uang and Nakashima (2003) provide a comprehensive treatment of different BRB concepts available worldwide. In the United States, BRB concepts commercially available to date, early 2004, have brace end connections that fall into one of the two types shown in Figure 3. In the United States, admissibility of a BRB concept for use in a building project is based on the BRB's meeting the acceptance criteria of section 8.6.3.7.10 of the 2003 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 450) (2004).

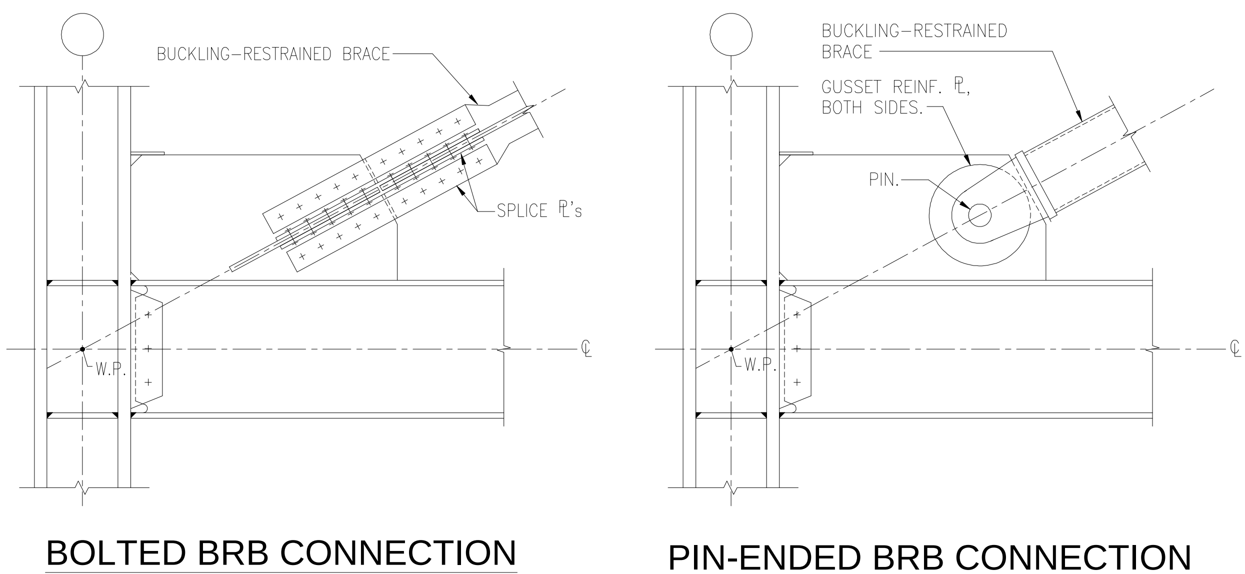


Figure 3. Two Types of BRB-to-Beam-Column Connections

In the United States, BRBFs are typically designed using an equivalent-lateral-force method. As in the typical design procedure employed for other concentrically braced-frame types, a linear elastic model is subjected to a reduced seismic load in order to determine the required strength and to verify adequate stiffness of the frame. For a BRBF with braces proportioned according to this method, the difference between the elastic and inelastic deformation modes is much less dramatic than for a Special Concentrically Braced Frame (SCBF). Because of this, an inelastic dynamic analysis is not typically required, although inelastic analyses give a much better estimate of brace ductility demands than elastic analyses (Fahnestock et al., 2003).

For such an elastic analysis to be valid, the brace element used in the analysis should correspond to tested brace behavior, and similarly, brace tests should corroborate the strength and ductility assumed in the analysis. Accordingly, BRBF design is based on the results of successful tests. Successful tests are those that exhibit full, stable hysteretic behavior with only moderate compression overstrength while achieving maximum and cumulative plastic ductility values in excess of those required by the actual building project.

Once BRBs have been designed for adequate strength, the adjoining frame elements are designed to the adjusted BRB strengths corresponding to $2.0\Delta_{bm}$ deformations ($1.0\Delta_{bm}$ for nonlinear dynamic analyses). These adjusted BRB strengths can be significantly higher than the brace design force due to oversizing of the brace, use of a resistance factor, compression overstrength, and, most significantly, strain hardening of the brace at large deformations and under repeated cyclic inelastic loading. This adjusted BRB strengths are determined from a backbone curve similar to that shown in Figure 4. It is worth noting that $2.0\Delta_{bm}$ is the value being considered for inclusion in the 2005 edition of the *AISC Seismic Provisions for Structural Steel Buildings (Seismic Provisions)* while $1.5\Delta_{bm}$ is the value published in FEMA 450. As explained in the following paragraph, where applicable, this *Steel TIPS* report will present the design requirements that correspond to the most up-to-date thinking on BRBFs as of July 2004.

The design of BRBFs is not yet governed by any building code. Recommended provisions for the design of BRBFs are available, however. A set of *Recommended Provisions for Buckling-Restrained Braced Frames* (*Recommended Provisions)* was developed by a joint AISC/SEAOC task group with the intention of including the provisions in the 2005 edition of the *Seismic Provisions*. The *Recommended Provisions* have been reviewed and have been included in Chapter 8 of FEMA 450. Currently the *Recommended Provisions* are being updated as the *Seismic Provisions* committee reviewing them generates comments. It is expected that the 2005 edition of the *Seismic Provisions* will adopt a more updated version of the *Recommended Provisions* than what was published in FEMA 450. The design example found in section 3 of this *Steel TIPS* report is based on the *Recommended Provisions* published in FEMA 450 with the updates proposed by the *Seismic Provisions* committee as of July 2004. The *Recommended Provisions* published in FEMA 450 include design procedures and detailed testing requirements for establishing the adequacy of BRBs.

Chapter 4 of FEMA 450 includes BRBF system factors R, C_d, $C_t(C_r)$, Ω_0, and x. It is expected that the *ASCE Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02) (2002) and model building codes will adopt the BRBF system factors found in FEMA 450 by reference. The beam-column connections of this *Steel TIPS* report are as shown in Figure 3, thus allowing the BRB frame system to use an R of 8. However, that does not imply that a dual system is being designed. The design example of this *Steel TIPS* report is not for a dual system.

Two types of brace tests are required by FEMA 450. The first is a uniaxial test that requires the BRB specimen to be of a similar size to those used in the actual building project. In this test a BRB specimen is loaded axially and cycled through the prescribed displacements until it has dissipated a minimum amount of energy. This test is intended to verify the adequacy of the BRB design using representative proportions.

The second type of brace test is called a subassemblage test. In this test, the BRB specimen is loaded axially while the end connections are rotated to simulate the conditions to be expected when BRBs are employed in a frame. This test is intended to verify that the brace-end rotational demands imposed by the frame action will not compromise the performance of the BRB. This test is not intended to test the performance of a frame.

BRBFs can have braces in any one of a number of configurations. Because there is no strength or stiffness degradation in the braces, and because the tension and compression strengths are almost equal, the single-diagonal configuration is permitted without any penalty. The single-diagonal configuration is an effective way to take advantage of the high strengths possible for BRBs. The V and inverted-V configurations are also popular for BRBFs, as they allow some openness in the frame. Because of the balance between brace tension and compression strength, the beam is required to resist modest loads in comparison to SCBFs; a deflection limit is also imposed to prevent excessive vertical beam displacement. Other BRBF configurations are possible.

2. SELECTED SUMMARY OF TESTS PERFORMED TO DATE

Numerous uniaxial and subassemblage tests have been performed on the different available BRBs. Results obtained from such tests can be thought of as falling into one of the following categories.

- Published results corresponding to tests performed in direct support of U.S. construction projects (Black et al., 2002; Merritt et al., 2003a, 2003b; SIE, 1999, 2001, 2003; UC Berkeley, 2002). Proprietary BRBs mentioned in the preceding references are Unbonded Braces™ manufactured by Nippon Steel Corporation (http://www.unbondedbrace.com/), buckling-restrained braces manufactured by CoreBrace (http://www.corebrace.com/), and PowerCat™ braces manufactured by Star Seismic (http://www.starseismic.net/). For access to the preceding reports and other unpublished reports, the structural engineer should contact the brace manufacturers directly.
- Published and unpublished results corresponding to the developmental testing phase of BRB concepts (Merritt et al., 2003c; Staker and Reaveley, 2002). The structural engineer should contact brace manufacturer directly for access to test results. For access to the Merritt et al., (2003c) report, the structural engineer should contact Associated Bracing directly at 510-583-5800.
- Published results corresponding to tests performed outside of the United States and not in direct support of U.S. construction projects. These published results are too many to mention, and their description is beyond the scope of this *Steel TIPS* report. The structural engineer is encouraged to consult Uang and Nakashima (2003) for a summary of these tests.

Because BRB concepts and their associated testing are too many to list and describe, Table 1 lists only those BRB concepts with public test results in support of actual U.S. building projects. In Table 1, ASTM refers to the American Society for Testing and Materials, and JIS refers to Japanese Industrial Standards.

2.1 BRB Backbone Curve (Strength Adjustment Factors)

One of the main derivations of test results is the BRB backbone curve. This curve is defined by the brace strain and normalized axial force. From the backbone curve, the engineer can extract the strength adjustment factors ω and $\omega\beta$ necessary for computing the adjusted BRB strengths. Figure 4 shows the backbone curve of an example BRB. During the design of an actual building project, the structural engineer calculates ω and $\omega\beta$ values from actual graphs supplied by the brace manufacturers being considered for the project and uses the more conservative values from the graphs.

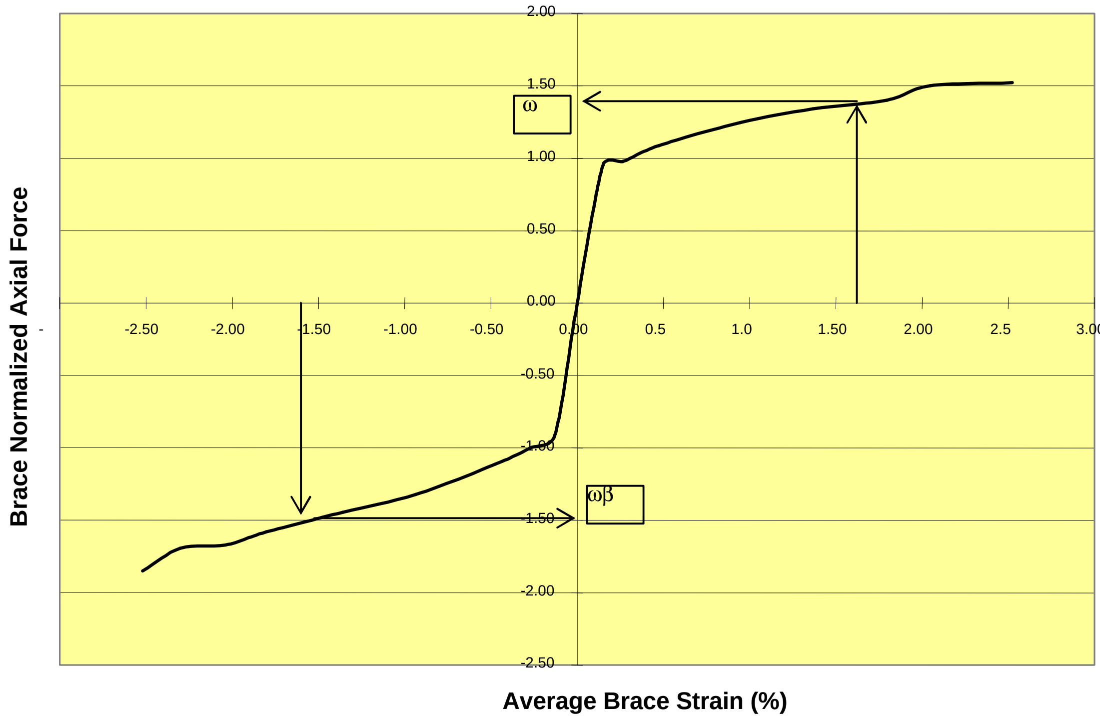


Figure 4. Backbone Curve of an Example BRB

Table 1. Selected BRB Tests

Year of Test	Literature Reference	Test Type	Number of Tested Braces	Steel Core Material	Tested Brace Sizes P_{ysc} (kip)	Brace Length (ft)	Max. Brace Strain[1] (%)	Max. Brace Ductility Demand[1] μ	Brace Cumulative Plastic Ductility[1] $\Sigma\mu_P$
1999	SIE, 1999	Uniaxial	3	JIS G3136 SM 490A	274	14.75	2.07	10	251
					365	14.75	2.07	10	251
					485	14.75	2.07	10	251
2001	SIE, 2001	Uniaxial	2	JIS G3136 SN 400B	457	14.75	2.07	15	345
					457	14.75	2.07	15	345
2002	UC Berkeley, 2002	Frame (Subassemblage)	3	JIS G3136 SN 400B	259	9.83	2.12	15	> 400[2]
					259	15.5	1.88	13	> 200[2]
					478	15.5	1.81	13	> 300[2]
2002	Merritt et al., 2003a	Subassemblage	6	ASTM A36	388	18	2.50	16	503
					388	18	2.50	16	495
					712	18	2.68	14	372
					712	18	2.62	13	368
					897	19	2.48	14	389
					897	19	2.40	14	384
2002	Merritt et al., 2003b	Subassemblage	8	ASTM A36	160	21	2.43	11	460
					250	21	2.48	11	460
					350	21	1.84	11	350
					500	21	2.47	11	400
					750	21	2.64	11	440
					750	21	2.54	11	440
					1200	21	1.84	11	310
					1200	21	1.77	11	325
2003	Merritt et al., 2003c	Uniaxial	2	ASTM A36	460	20	1.60	8	158
					460	20	1.72	9	174
2003	SIE, 2003	Subassemblage	4	JIS G3136 SN 400B	783	13.85	2.73	17	513
					783	24.78	1.64	11	288
					1162	13.85	2.96	18	584
					1162	24.78	1.63	11	308

Notes:

[1] Values are as reported in the literature for the normal displacement protocol. Values exclude results from supplemental or fatigue tests where applicable.

[2] Values indirectly obtained. Test setup did not lend itself to direct determination of brace demands.

3. SEVEN-STORY OFFICE BUILDING EXAMPLE

This section illustrates the procedure for designing a BRBF building using the loading demands prescribed in ASCE 7-02 and performing the design checks utilizing the Section 8.6 of Chapter 8 of FEMA 450. A copy of FEMA 450 can be downloaded from the Building Seismic Safety Council's website at http://www.bssconline.org/. Before proceeding with this example, the reader is highly encouraged to obtain a copy of chapter 8 of FEMA 450.

3.1 Project Information

The building considered has the same total height and seismic weight as that of *Steel TIPS* reports published in November 1995 and December 1996; namely, "Seismic Design of Special Concentrically Braced Frames" and "Seismic Design Practice for Eccentrically Braced Frames." While the site seismicity and seismic load resisting system are different for this *Steel TIPS* report, the use of the same building model is intended to provide a point of reference for comparison of different braced-frame systems. Figures 5 and 6 define the building and system geometries.

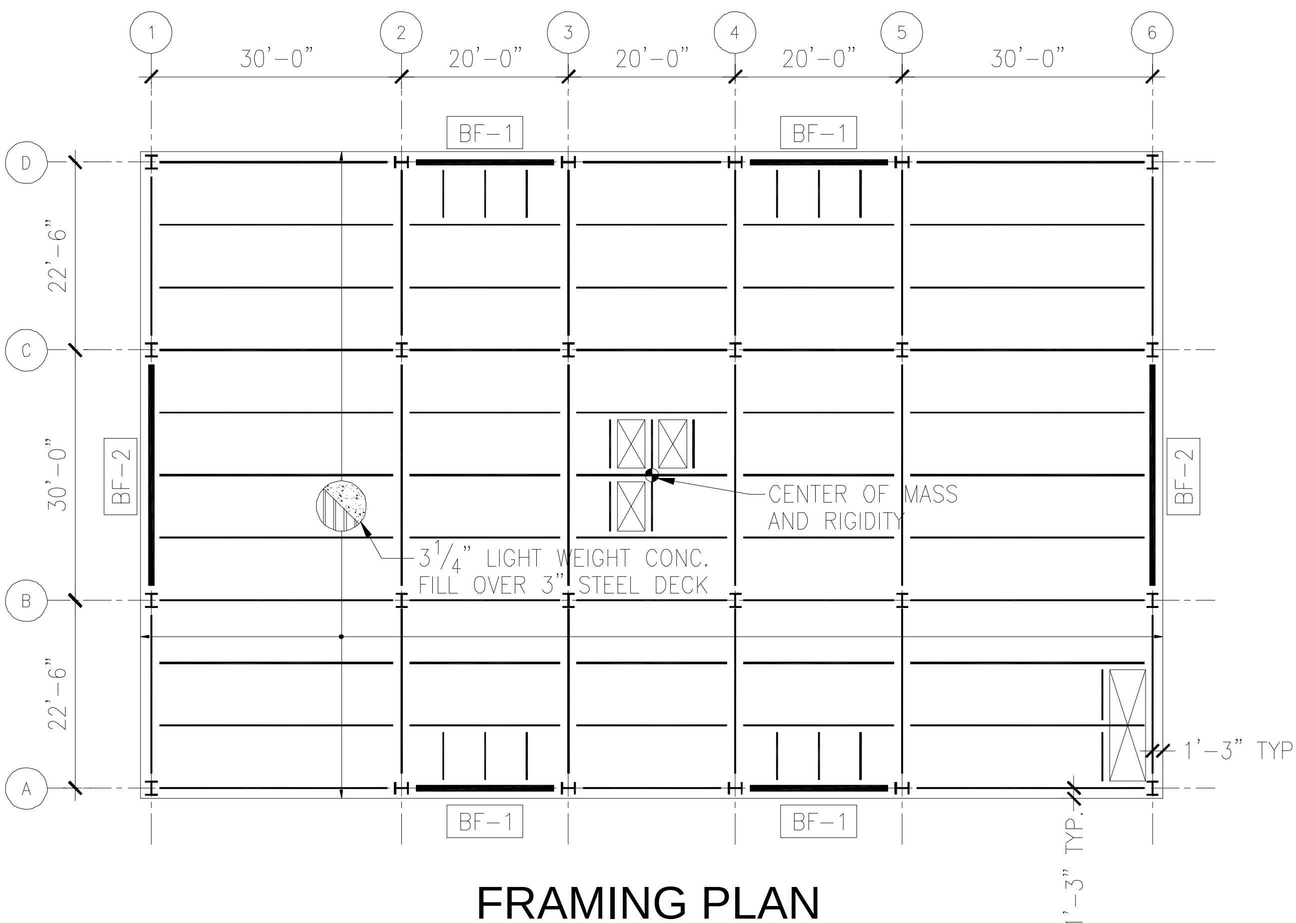
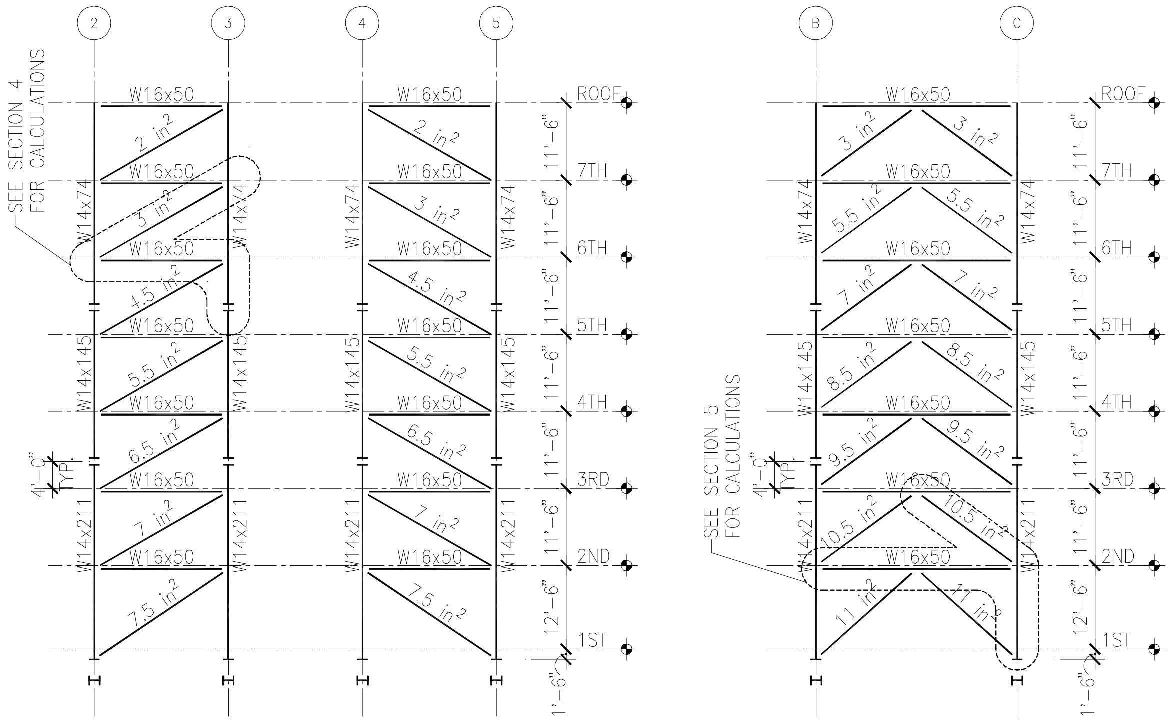


Figure 5. Framing Plan



TYPICAL BF-1 ELEVATION   TYPICAL BF-2 ELEVATION

Figure 6. BRBF Elevations

Notes: 1. Parapet skin extending 2'-0" above roof slab is not shown.
2. XX in^2 denotes A_{sc}, steel core area of the BRB.

<u>Structural Materials</u>

W sections	ASTM A992 (F_y = 50 ksi, F_u = 65 ksi)
BRB Steel Core	ASTM A36 or JIS G3136 SN 400B with supplemental yield requirements: F_{ysc} =42 ksi (±4ksi). Coupon tests required.
BRB Steel Casing	ASTM A500 Grade B or JIS G 3466 STKR 400
Gusset plates	ASTM A572, Grade 50 (F_{yg} = 50 ksi, F_u = 65 ksi)
Weld electrodes	E70XX (notch toughness: 20 ft-lb at –20 degrees Fahrenheit)
Lightweight concrete fill	f_c' = 3000 psi

Since either bolts or a pin can be used to connect the brace to the gusset, specifications for both are provided

High strength bolts (if used)	ASTM A325 or A490 SC Design note: use of factored load design strengths is encouraged to reduce connection length and costs.
Pins (if used)	ASTM A354 Grade BC round stock Design note: pin connections should comply with AISC *Load and Resistance Factor Design Manual of Steel Construction* (AISC LRFD) (2001) Specification D3

Loading

Roof Loading:

Roofing and insulation	7.0	psf
Steel deck + Fill	47.0	
Steel framing and fireproofing	8.0	
Ceiling	3.0	
Mechanical/Electrical	2.0	
Total	67.0	psf

Note that to be consistent with a previous issue of *Steel TIPS* report, the partition wall contribution to the roof's seismic weight is not accounted for. The structural engineer must decide on a project by project basis whether to include a portion of the partition load in the seismic weight calculations.

Floor Weights:

Steel deck + Fill	47.0	psf
Steel framing and fireproofing	13.0	
Partition walls	20.0	
Ceiling	3.0	
Mechanical/Electrical	2.0	
Total	85.0	psf

Average Exterior Curtain Wall Weight
including Column and Spandrel Covers: 15.0 psf

Live Loads:

Roof	20	psf
Floor	50	psf

Site Seismicity

Assume that the building project is located in the San Francisco Bay Area in a site with latitude and longitude such that the soil is classified as type D, F_a = 1.0, F_v = 1.5, and the Maximum Credible Earthquake (MCE) parameters given in Table 2 are obtained.

Table 2. Site Parameters

MCE		MCE with soil factors	Design S_a
Period (sec)	S (g)	S_M (g)	S_D (g)
T = 0.2	1.541	1.541	1.027
T = 1.0	0.887	1.331	0.887

The response spectrum is constructed per section 9.4.1.2.6 of ASCE 7-02 and shown in Figure 7. Throughout this report all equations and section references are for ASCE 7-02 unless otherwise noted.

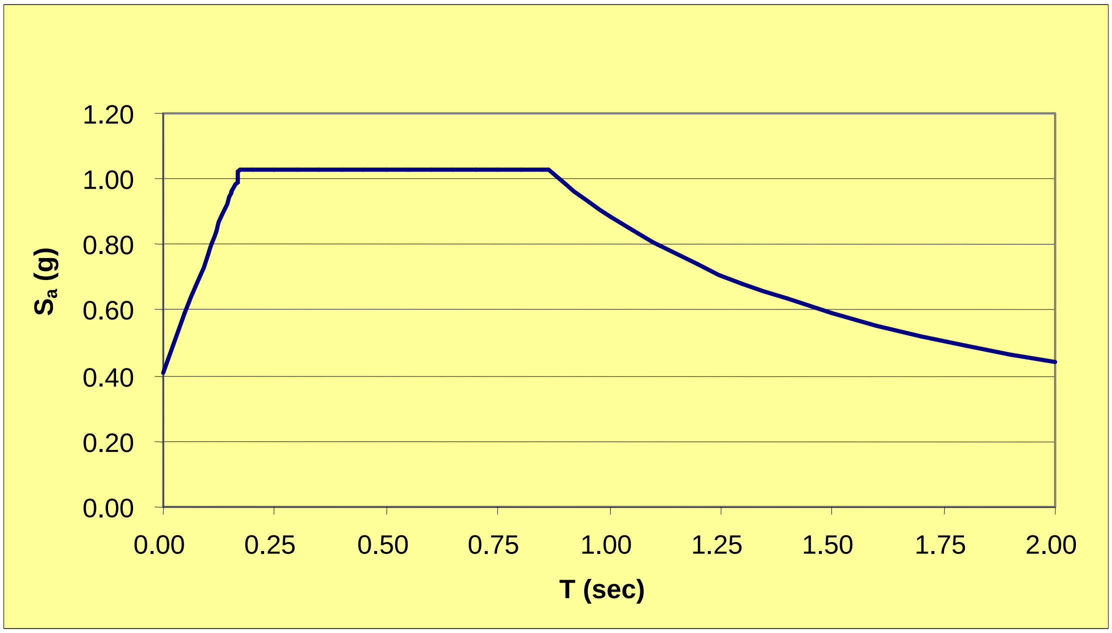


Figure 7. Design Response Spectrum

Seismic Load Resisting System Parameters

The values of R, C_d, C_t (C_r), and x listed in Table 3 are found in Chapter 4 of FEMA 450.

Table 3. System Parameters

Parameter	Value	Reference
Building Height	83 ft	Per Elevation
Occupancy Category	II	Table 1-1
Seismic Use Group	I	Table 9.1.3
Seismic Design Category	E	Table 9.4.2.1a,b
Importance Factor, I	1.0	Table 9.1.4
Seismic Weight (W)	5,931 kips	Definition
Seismic Load Resisting System	BRBF with moment-resisting beam-column connections	Definition
R	8.0	FEMA 450
C_d	5	FEMA 450
C_t (C_r)	0.03	FEMA 450
x	0.75	FEMA 450
C_u	1.4	Table 9.5.5.3.1

3.2 Seismic Force Computation

Fundamental Period (9.5.5.3)

Period, Ta: $T_a = C_t \cdot h_n^{\,x}$ (Eq. 9.5.5.3.2-1)

not to exceed: $T = C_u \cdot T_a$ (Table 9.5.5.3.1)

For this example
T_a = 0.82 sec
When calculating C_s, the actual period of the structure (T) cannot be taken greater than 1.15 sec

Base Shear (9.5.5.2)

Base Shear, V: $V = C_s \cdot W$ (Eq. 9.5.5.2-1)

$$C_s = \frac{S_{DS}}{\frac{R}{I}}$$ (Eq. 9.5.5.2.1-1)

Cs should not exceed: $$C_s = \frac{S_{D1}}{T \cdot \left(\frac{R}{I}\right)}$$ (Eq. 9.5.5.2.1-2)

Cs should not be less than: $C_s = 0.044 \cdot I \cdot S_{DS}$ (Eq. 9.5.5.2.1-3)

Cs should not be less than (seismic design categories E & F): $$C_s = \frac{0.5 \cdot S_1}{\frac{R}{I}}$$ (Eq. 9.5.5.2.1-4)

Story Force (9.5.5.4)

Force at each level: $F_x = C_{vx} \cdot V$ (Eq. 9.5.5.4-1)

$$C_{vx} = \frac{w_x \cdot h_x^{\,k}}{\sum_{i=1}^{n} w_i \cdot h_i^{\,k}}$$ (Eq. 9.5.5.4-2)

where: k is linearly interpolated between 1 and 2 for structures having period between 0.5 and 2.5 sec.

k = 1.16 for this example

Story Shear (9.5.5.5)

$$V_x = \sum_{i=x}^{n} F_i$$ (Eq. 9.5.5.5)

Using the preceding formulas, we are able to compute:

C_s = 0.128
C_s ≤ 0.134
C_s = 0.045
C_s = 0.055 (for seismic categories E and F)

Therefore, $V = C_sW = 0.128W = 761.4$ kips. See Table 4 for seismic force distribution values.

Table 4. Seismic Force Distribution

Level	w_i (kips)	h_i (ft)	$w_i \times h_i^{\ k}$ (kip-ft)	C_{vx}	Story Force F_x (kip)	Story Shear V_x (kip)	Overturning Moment O.M. (kip-ft)
Roof	687	83	115,634	0.217	165		
7^{th}	874	72	123,739	0.232	177	165	1,900
6^{th}	874	60	100,964	0.189	144	342	5,833
5^{th}	874	49	78,881	0.148	113	486	11,424
4^{th}	874	37	57,627	0.108	82	599	18,312
3^{rd}	874	26	37,420	0.070	53	681	26,147
2^{nd}	874	14	18,665	0.035	27	735	34,596
1^{st}	-	-	-		-	761	45,256
Total	5,931		532,929	1.000	761		

3.3 Building Seismic Load Analysis and Determination of Demands

The analysis procedures followed in this example depart from previous *Steel TIPS* reports in that:

- It allows the design story shear to be shared between the braces and the braced frame columns in proportion to their relative rigidities. We again note that this design example is not of a dual system but of a BRBF system detailed with moment-resisting beam-column connections so that an R = 8 can be used.
- It explicitly accounts for the braced frame column base fixity created by both gusset plates and the need to resist large uplift forces. Because of the number of bays used to resist seismic loads and the capacity design approach prescribed in the *Recommended Provisions*, BRBF columns resist high tensile loads. As a result, complete joint penetration welds and thick plates are normally specified at the column base. The as-detailed column-base connection consists of a column fully welded to a thick base plate, with a vertical gusset stiffening the joint. Therefore, it seems appropriate to acknowledge the fixity of the column base when performing the analysis. The moment generated at the column base will be resisted by a concrete-compression anchor-rod-tension couple. The shear generated at the column base will be resisted by steel elements (angles, plates, rebar) parallel to the frame and welded to the top of the base plate allowing the anchor rods to resist tension only.

Because of these procedures, simple truss-force models are not sufficient, and a model that includes flexural properties is required. Accordingly, a computer model is used, and enough information and results are shown so that the reader can follow the presentation of analysis and design recommendations.

This departure from other examples should not represent a shortcoming for the S*teel TIPS* report reader, since the focus is on describing how to perform the design of a relatively new braced-frame system.

Computer Model Description

Following is a description of the computer model.

- For simplicity, there is no distinction between roof and floor live load. All live load is modeled as floor live load.
- For simplicity, live load is not reduced.
- In computing uniform dead and live loads applied on the frame beams, the loading corresponding to the 1'-3" tributary edge of the slab has been neglected for simplicity.
- Self-weight is not calculated by the computer program.
- It is assumed that appropriately factored wind loading is smaller than the seismic base shear computed in Table 4 and that its heightwise distribution does not cause yielding of the BRBs.
- Braces are modeled as pin-ended.
- As shown in Figure 8, the actual length of the steel core is smaller than the work-point-to-work-point length of the brace. As a result, the actual stiffness of the brace is greater than that computed using only the steel core area. For this example, the effective stiffness of the BRB is defined as 1.4 times the stiffness computed using only the steel core. This is consistent with many actual designs.
- In order to provide a conservative brace design, the beams were assigned no rigid offset length at their connections.
- Floor diaphragms are modeled as rigid.
- To determine the axial loads in the BF-1 frame beams, frame nodes along lines 3 and 4 and along lines A and D were disconnected from all floor diaphragms.
- To determine the axial loads in the BF-2 frame beams, frame nodes along line B.5 and along lines 1 and 6 were disconnected from all floor diaphragms.
- Seismic forces were applied at the center of mass at each diaphragm as point loads. In addition, a moment was applied to account for accidental torsion (5% eccentricity).
- Frame columns are modeled as fixed at their bases. See the previous section for an explanation and a description of the base detail.

Calculation of Load Factor Rho (ρ)

The mechanics of calculating the rho factor (ρ), a load factor, is covered sufficiently in other literature (SEAOC, 1999) and will not be repeated here. When calculating the rho factor, the portion of the story shear resisted by the braces is that which the braces resist in proportion to their stiffness compared to the stiffness of the frame surrounding the braces.

After performing analyses in both building directions and computing rho throughout the height of the building, the worst case rho factor is chosen for each direction. Table 5 summarizes the results.

Table 5. Rho Factor

Building Direction	A_x (sq. ft)	Governing Story	r_{max}	ρ_{max}
X-direction	9,000	3rd	0.236	1.11
Y-direction	9,000	2nd	0.272	1.23

Applicable Load Combinations

With the calculated $\rho_X = 1.11$, $\rho_Y = 1.23$, and $0.25S_{DS}D = 0.21D$ expansion of equations (9.5.2.7-1) and (9.5.2.7-2) into load combination 5 and 7 of section 2.3.2 in ASCE 7-02 gives the following sixteen load combinations defining the required strengths of BRBs, frame beams, and frame columns associated with the seismic base shear.

LC1:	$1.41D + 0.5L + 1.11*POSECCEQ_X$
LC2:	$1.41D + 0.5L - 1.11*POSECCEQ_X$
LC3:	$0.69D + 1.11*POSECCEQ_X$
LC4:	$0.69D - 1.11*POSECCEQ_X$
LC5:	$1.41D + 0.5L + 1.11*NEGECCEQ_X$
LC6:	$1.41D + 0.5L - 1.11*NEGECCEQ_X$
LC7:	$0.69D + 1.11*NEGECCEQ_X$
LC8:	$0.69D - 1.11*NEGECCEQ_X$
LC9:	$1.41D + 0.5L + 1.23*POSECCEQ_Y$
LC10:	$1.41D + 0.5L - 1.23*POSECCEQ_Y$
LC11:	$0.69D + 1.23*POSECCEQ_Y$
LC12:	$0.69D - 1.23*POSECCEQ_Y$
LC13:	$1.41D + 0.5L + 1.23*NEGECCEQ_Y$
LC14:	$1.41D + 0.5L - 1.23*NEGECCEQ_Y$
LC15:	$0.69D + 1.23*NEGECCEQ_Y$
LC16:	$0.69D - 1.23*NEGECCEQ_Y$

Where:
$POSECCEQ_X$ = EQ_X with 5% positive eccentricity.
$NEGECCEQ_X$ = EQ_X with 5% negative eccentricity.
$POSECCEQ_Y$ = EQ_Y with 5% positive eccentricity.
$NEGECCEQ_Y$ = EQ_Y with 5% negative eccentricity.

And the eccentricities for the applied seismic base shear are as follows:
EQ_X: e_Y = 0.05 x 75' = 3.75'
EQ_Y: e_X = 0.05 x 120' = 6'

Calculation of Design Story Drifts

The above sixteen strength load combinations were modified by setting $\rho_X = 1$ and $\rho_Y = 1$ and then were used to calculate interstory drift ratios. The exclusion of rho in calculating drift is explicitly described in Section 9.5.5.7.1 of ASCE 7-02. Alternatively, it is possible to calculate the actual period of the structure and use it to calculate a reduced base shear for drift computation. However, such an approach is not followed here. Taking advantage of reduced base shear for drift computation is advantageous in the design of building structures that are either taller than the building in this example or more sensitive to drift demands.

The procedure followed to calculate design story drifts entailed calculating elastic story deflections for the load combinations resulting in the largest deflections. Then, elastic story drifts, Δ_x, were calculated as the difference of the deflections at the top and bottom of the story under consideration. Then, design story drifts, Δ, were calculated as the product of Δ_x and C_d divided by I. Δ_M is another term for design story drift introduced for ease of pairing the BRB axial deformation Δ_{bm} to Δ_M. Utilizing the C_d and I values defined in Table 3, the design story drifts for BF-1 and BF-2 frames were computed and are summarized in Tables 6 and 7.

Where:
h_{sx} = defined in Section 9.2.2
Δ_x = elastic story drift = defined by (Eq. 9.5.5.7.1)
Δ = Δ_M = defined in Section 9.5.5.7.1
Δ_a = defined per Table 9.5.2.8
θ_x = interstory drift ratio from elastic analyses. This definition is similar to that shown in Section S2 of the *Seismic Provisions*.
θ_M = $\Delta_M \div h_{sx}$
θ_a = $\Delta_a \div h_{sx}$
See Figure 9 for an illustration of the preceding definitions.

Table 6. Design Story Drifts for BF-1 Frames

Story	Story Height h_{sx} (in)	Elastic Story Drift Δ_x (in)	Design Story Drift $\Delta = \Delta_M$ (in)	Allowable Story Drift Δ_a (in)	Interstory Drift Ratio θ_x (%)	Design Drift Ratio θ_M (%)	Allowable Drift Ratio θ_a (%)
7th	138	0.39	1.97	2.76	0.29	1.43	2.00
6th	138	0.47	2.37	2.76	0.34	1.72	2.00
5th	138	0.48	2.38	2.76	0.34	1.72	2.00
4th	138	0.46	2.29	2.76	0.33	1.66	2.00
3rd	138	0.43	2.16	2.76	0.31	1.57	2.00
2nd	138	0.39	1.97	2.76	0.29	1.43	2.00
1st	168	0.35	1.74	3.36	0.21	1.03	2.00

Table 7. Design Story Drifts for BF-2 Frames

Story	Story Height h_{sx} (in)	Elastic Story Drift Δ_x (in)	Design Story Drift $\Delta = \Delta_M$ (in)	Allowable Story Drift Δ_a (in)	Interstory Drift Ratio θ_x (%)	Design Drift Ratio θ_M (%)	Allowable Drift Ratio θ_a (%)
7th	138	0.51	2.54	2.76	0.37	1.84	2.00
6th	138	0.50	2.50	2.76	0.36	1.81	2.00
5th	138	0.44	2.21	2.76	0.32	1.60	2.00
4th	138	0.40	2.00	2.76	0.29	1.45	2.00
3rd	138	0.38	1.92	2.76	0.28	1.39	2.00
2nd	138	0.30	1.48	2.76	0.21	1.07	2.00
1st	168	0.27	1.33	3.36	0.16	0.79	2.00

4. DESIGN OF SINGLE-DIAGONAL BRACED FRAME

4.1 Brace Demands and Brace Capacities

This section illustrates the design of the 6th story BRB along line A between lines 2 and 3. See Figure 6. First, the brace required strength is calculated utilizing the computer run results:

$\rho_X = 1.11$
$P_E = 85.82$ kips (POSECCEQ$_x$)
$P_D = 0.79$ kips
$P_L = 0.48$ kips
LC1: $P_u = 1.41\,P_D + 0.5\,P_L + \rho_X\,P_E$
$P_u = 96.6$ kips

Then the design strength is calculated taking into account material variability. The material specifications for this example require an average F_{ysc} = 42 ksi with a tolerance of ± 4 ksi. Since the steel core areas shown in Figure 6 are the minimum required to comply with drift provisions, F_{ysc} variability is accounted for by using the lowest permissible F_{ysc} (38 ksi) when calculating BRB design strengths and the largest permissible F_{ysc} (46 ksi) when calculating adjusted BRB strengths.

For the BRB DCRs to meet the *Recommended Provisions* requirements,

$\phi P_n = \phi P_{ysc} = \phi F_{ysc}\,A_{sc}$
$= 0.9 \times 38 \text{ ksi} \times 3 \text{ in}^2 = 102.6$ kips

(DCR) $= \dfrac{P_u}{fP_n} = \dfrac{96.6}{102.6} = 0.94 < 1.00$ OK

If steel core supplied with F_{ysc} = 42 ksi, DCR = 0.85 < 1.00 OK

If steel core supplied with F_{ysc} = 46 ksi, DCR = 0.78 < 1.00 OK

4.2 Computation of 2.0D$_{bm}$, Brace Strains, and Adjusted Brace Strengths

Per FEMA 450 section 8.6.3.2.2.2, brace strains associated with 1.5Δ_{bm} need to be within the range of strains that have been successfully tested. Note that if a nonlinear dynamic analysis procedure had been chosen, the required Δ_{bm} computation would have been only 1.0Δ_{bm} (FEMA 450 commentary section 8.6.3.1). An update to the FEMA 450 value of 1.5Δ_{bm} is the 2.0Δ_{bm} value being considered for inclusion in the 2005 edition of the *Seismic Provisions*. As with the 2.0Δ_{bm} proposed update, where applicable, this S*teel TIPS* report will present the latest thinking on BRBF that is likely to be included in the 2005 edition of the *Seismic Provision*.

The steps associated with this section are as follows. See Table 8.

- For the load combination producing the largest elastic story drift, Δ_x, extract from the computer program the corresponding axial load, P_{bx}. Because load combinations used to calculate story drifts utilize $\rho_X = \rho_Y = 1$, P_{bx} is less than the required axial strength, P_u.
- Estimate BRB yield length, L_{ysc}. See Figure 8. Since BRB yield length varies with brace manufacturer, the structural engineer should obtain length estimates from the manufacturer prior to calculating the BRB strains. For this example, it is assumed that after sizing the braces for strength, a brace manufacturer was given enough information to determine that for BF-1 BRBs the yield length can be approximated as two thirds of the work-point-to-work-point length, $L_{ysc} = 0.66L_1$. Consultation with a brace manufacturer early on in the design process ensures obtaining information accurate enough to prevent the need for recalculating interstory drift ratios, brace strains, and brace adjusted strengths during the submittal review phase.
- Compute the BRB axial deformation corresponding to the elastic story drift, Δ_{bx}.

  $\Delta_{bx} = \frac{P_{bx}L_{ysc}}{EA_{sc}}$, where

  E = 29000 ksi

  A_{sc} = steel core area defined in Figure 6
- Compute the BRB axial deformation corresponding to the design story drift, Δ_{bm}.

  $\Delta_{bm} = C_d \Delta_{bx}$
- Compute the average brace strain, ε_{BRC}.

  $e_{BRC} = \frac{2.0?_{\ bm}}{L_{ysc}}$
- Once the brace strains are calculated, compute strength adjustment factors, ω and $\omega\beta$, from the backbone curve derived from the test results. For this example, our backbone curve is defined as that shown in Figure 4.
- Compute adjusted BRB strengths, T_{MAX} and C_{MAX}, using the upper-bound yield strength allowed by the material specifications, F_{ysc} = 46 ksi for this example.

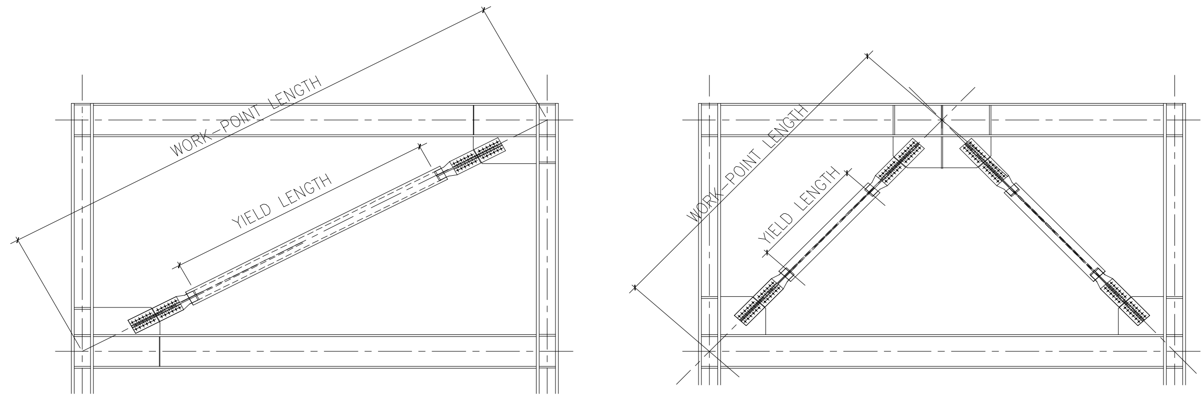


Figure 8. Illustration of BRB Yield Lengths

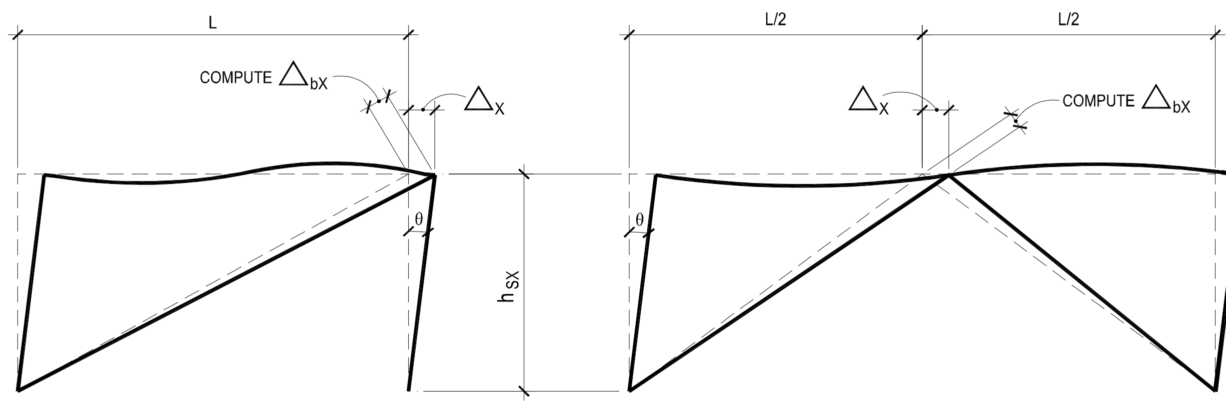


Figure 9. BRB Deformation

Table 8. Strength Adjustment Factors for BF-1 BRBs

Story	A_{sc} (sq. in)	P_{bx} (k)	L_{ysc} (in)	Δ_{bx} (in)	Δ_{bm} (in)	$2.0\Delta_{bm}$ (in)	ε_{BRC} (%)	Adjustment Factors		
								ω	ωβ	β
7th	2.00	38.3	184.5	0.12	0.61	1.22	0.66	1.12	1.14	1.01
6th	3.00	84.5	184.5	0.18	0.90	1.79	0.98	1.22	1.25	1.03
5th	4.50	132.3	184.5	0.19	0.94	1.87	1.02	1.23	1.27	1.03
4th	5.50	159.8	184.5	0.18	0.92	1.85	1.01	1.22	1.27	1.03
3rd	6.50	188.9	184.5	0.18	0.92	1.85	1.01	1.22	1.27	1.03
2nd	7.00	201.6	184.5	0.18	0.92	1.83	1.00	1.22	1.26	1.03
1st	7.50	198.8	195.3	0.18	0.89	1.78	0.92	1.20	1.23	1.03

Table 9. Adjusted BRB Strengths for BF-1 Frames

Story		F_{ysc}=46 ksi		
	A_{sc} (sq. in)	P_{ysc} (k)	T_{MAX} ωP_{ysc} (k)	C_{MAX} $\omega\beta P_{ysc}$ (k)
7th	2.00	92.00	103	105
6th	3.00	138.00	168	173
5th	4.50	207.00	254	263
4th	5.50	253.00	310	320
3rd	6.50	299.00	366	378
2nd	7.00	322.00	393	406
1st	7.50	345.00	413	425

where $P_{ysc} = F_{ysc}A_{sc}$

4.3 Beam Design

This section illustrates the design of the 6th floor beam on line A between lines 2 and 3. See Figure 6. The design is performed in two stages.

- Check beam design strengths against the required axial, flexural, and shear strengths associated with the seismic base shear (LC1 through LC16). See section 4.3.1.
- Check beam axial design strengths against the required axial strength induced by the adjusted BRB strengths at $2.0\Delta_{bm}$ ($1.0\Delta_{bm}$ for nonlinear dynamic analyses). Adjusted BRB strengths, T_{MAX} and C_{MAX}, are shown in Table 9. The T_{MAX} values are the governing BRB strengths because they produce compressive forces in the beams. The C_{MAX} values produce higher tensile forces in the beams and do not govern. See section 4.3.2.

4.3.1 Design Check to Required Strengths Induced by the Seismic Base Shear

The required axial, flexural, and shear strengths are first extracted from the computer model, and then the beam design strengths are hand-calculated.

Required strength for load combination LC1:

$M_u := 96.8$ kip − ft $\quad V_u := 15.07$ kip $\quad P_u := 123.7$ kip

Trial section: Beam_Size := "W16X50"

$E := 29000$ ksi $\quad F_y := 50$ ksi $\quad \phi_b := 0.9 \quad L := 240$ in

$A_g = 14.7$ in^2 $\quad r_x = 6.68$ in $\quad r_y = 1.59$ in $\quad Z_x = 92$ in^3

$d = 16.26$ in $\quad t_w = 0.38$ in

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 5.62$ ft $\quad L_r = 15.7$ ft $\quad M_r = 270$ kip − ft

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.6.1 (Seismic Provisions, Table I-8-1)

flange: $\lambda_{ps} := 0.3 \cdot \sqrt{\dfrac{E}{F_y}} \qquad \lambda_{ps} = 7.22 \qquad \dfrac{b_f}{2 \cdot t_f} = 5.61 \qquad \dfrac{b_f}{2 \cdot t_f} < \lambda_{ps}$ OK

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.19$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 57.8 \qquad \frac{h}{t_w} = 37.4 \qquad \frac{h}{t_w} < \lambda_{ps} \quad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$l_x := L \qquad l_y := \frac{L}{4} \qquad k := 1.0$ (ky = 1.0, k x < 1.0. Use 1.0 as k y governs)

$\lambda_{c1} := \frac{k \cdot l_x}{\pi \cdot r_x} \cdot \sqrt{\frac{F_y}{E}} \qquad \lambda_{c1} = 0.47 \qquad \lambda_{c2} := \frac{k \cdot l_y}{\pi \cdot r_y} \cdot \sqrt{\frac{F_y}{E}} \qquad \lambda_{c2} = 0.5$

$\lambda_c := \max(\lambda_{c1}, \lambda_{c2}) \qquad \lambda_c = 0.5$

$F_{cr} := 0.658^{\lambda_c^2} \cdot F_y \qquad \lambda_c \le 1.5$ (E2-2)

$F_{cr} = 45.06$ ksi

$P_n := F_{cr} \cdot A_g$ (E2-1) $\qquad \phi_c := 0.85$ (E2-1)

$\phi_c \cdot P_n = 562.97$ kips

$\frac{P_u}{\phi_c \cdot P_n} = 0.22$

Bending Capacity (AISC LRFD section 16, Chapter F)

Beam is braced at quarter points

$L_b := \frac{20}{4} \qquad L_b = 5$ ft $\qquad L_p = 5.62$ ft

$M_p := F_y \cdot \frac{Z_x}{12}$ (F1-1) $\qquad M_p = 383.33$ kip – ft

$M_n := M_p \qquad L_b \le L_p$ (F1-1)

$\phi_b \cdot M_n = 345$ kip – ft

$\frac{M_u}{\phi_b \cdot M_n} = 0.28$

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v := 0.9 \qquad A_w := d \cdot t_w$ (F.2.1)

$V_n := 0.6 \cdot F_y \cdot A_w \qquad \frac{h}{t_w} \le 2.45 \cdot \sqrt{\frac{E}{F_y}}$ (F2-1)

$\phi_v \cdot V_n = 166.83$ kips

$\frac{V_u}{\phi_v \cdot V_n} = 0.09$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$R := \frac{P_u}{2 \cdot \phi_c \cdot P_n} + \frac{M_u}{\phi_b \cdot M_n} \qquad \frac{P_u}{\phi_c \cdot P_n} < 0.2$ (H1-1b)

Demand Capacity Ratio: $R = 0.39$ W16x50 OK

4.3.2 Design Check to Required Axial Strengths Induced by Deformations at 2.0D_{bm}

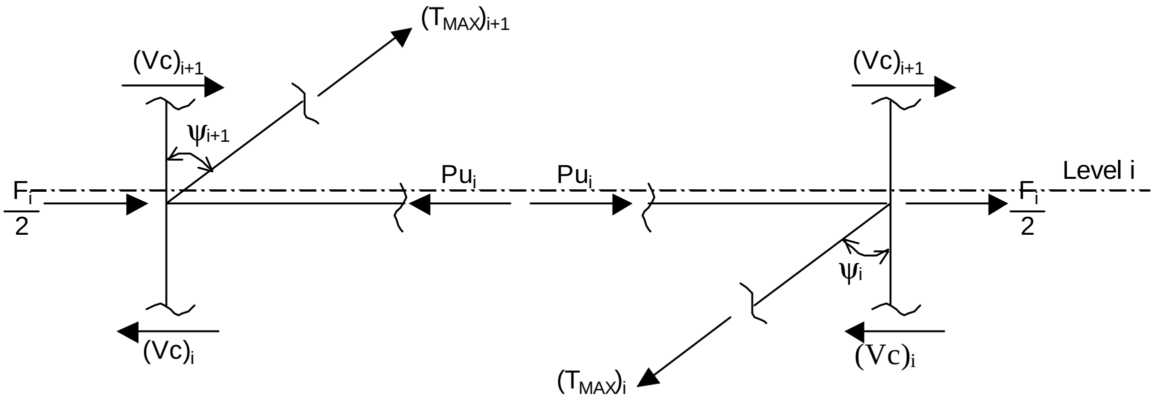


Figure 10. Required Axial Strengths of Sixth-Floor Beam per FEMA 450 Section 8.6.3.6.2

Since only an elastic analysis is performed, certain assumptions must be made to compute the axial force in the frame beam. These produce conservative results. They are:

- $(Vc)_{i+1}=(Vc)=0$ Shears in columns are assumed to be zero
- Fi is the sum of story collector forces corresponding to the mechanism under consideration. Collector forces are assumed equal at each end of the frame.

Alternatively, nonlinear analyses may be performed from which the actual demands in the members can be extracted.
Then,

$$F_i := TMAX_i \cdot \sin(\psi_i) - TMAX_{i+1} \sin(\psi_{i+1})$$

$$Pu_i := TMAX_{i+1} \cdot \sin(\psi_{i+1}) + \frac{F_i}{2}$$

$$P_u := Pu_i$$

For this example, level "i"=6th floor. From Table 9:

$TMAX_{i+1} := 168$ k $\quad \psi_{i+1} := 60.1$ deg (6th story)

$TMAX_i := 254$ k $\quad \psi_i := 60.1$ deg (5th story)

And,

$P_u = 183$ kips

Mu and Vu were obtained from the computer model due to factored vertical loads: 1.41D+0.5L.

$M_u := 26.2$ kip – ft $V_u := 8.1$ kips $P_u = 183$ kips

Trial section: Beam_Size := "W16X50"

$E := 29000$ ksi $F_y := 50$ ksi $\phi_b := 0.9$ $L := 20 \cdot 12$ in

$A_g = 14.7$ in^2 $r_x = 6.68$ in $r_y = 1.59$ in $Z_x = 92$ in^3

$d = 16.26$ in $t_w = 0.38$ in

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 5.62$ ft $L_r = 15.7$ ft $M_r = 270$ kip – ft

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.6.1 (Seismic Provisions, Table I-8-1)

At this higher force level, the compactness of the web must be reexamined. Pu is taken as that corresponding to a deformation of 2.0Δbm -- the BRBF equivalent of the amplified seismic load. This approach is chosen to meet the intent of the *Recommended Provisions,* which permit flexural yielding of the frame beams but do not allow for compression instability at these high axial forces. As more specific criteria for cyclic stability are developed, the following equation should be revised if needed.

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.28$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \qquad \lambda_{ps} = 55.39 \qquad \frac{h}{t_w} = 37.4 \qquad \frac{h}{t_w} < \lambda_{ps} \qquad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$\phi_c \cdot P_n = 562.97$ kips

$\dfrac{P_u}{\phi_c \cdot P_n} = 0.32$

Bending Capacity (AISC LRFD section 16, Chapter F)

Beam is braced at quarter points

$\phi_b \cdot M_n = 345$ kip – ft

$\dfrac{M_u}{\phi_b \cdot M_n} = 0.08$

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v \cdot V_n = 166.83$ kips

$$\frac{V_u}{\phi_v \cdot V_n} = 0.05$$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$$R := \frac{P_u}{\phi_c \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_u}{\phi_b \cdot M_n} \right) \qquad \frac{P_u}{\phi_c \cdot P_n} \geq 0.2 \qquad \text{(H1-1a)}$$

Demand Capacity Ratio: $R = 0.39$ W16x50 OK

4.4 Column Design

This section illustrates the design of column A/3 between the 5th and 6th floors. See Figure 6. The design is performed in two stages.

- Check column design strengths against the required axial, flexural, and shear strengths associated with the seismic base shear (LC1 through LC16). See section 4.4.1.
- Check axial design strengths against the required axial strength induced by the adjusted BRB strengths at $2.0\Delta_{bm}$. FEMA 450 section 8.6.3.5.3 requires that columns be designed to resist axial forces determined from the adjusted strengths of all connected BRBs. That is, the required axial strength of a column in a BRBF is the sum of the vertical components of the adjusted, strain-hardened capacity of all connected BRBs. This capacity-design requirement is equivalent to the one for columns in Eccentrically Braced Frames, and is based on the assumption of first-mode response of the structure. To the degree that higher modes participate in the seismic response of a building, the demands on BRBF columns can be expected to be lower than those prescribed in FEMA 450. Therefore, the requirement for capacity design of BRBF columns may be appropriate for lower buildings, but on the conservative side for taller ones, which tend to have greater participation from higher modes. While it is clear that for tall buildings the requirement may result in significant column overdesign, an accepted, straightforward method of estimating column demands has not yet been established, and in the interim, capacity-design procedures are required for these elements. Adjusted BRB strengths, T_{MAX} and C_{MAX}, are shown in Table 9 and in Figure 11. The T_{MAX} forces are the governing BRB forces because they produce compressive forces in column A/3. The C_{MAX} forces produce higher tensile forces in column A/3 and do not govern column design. See section 4.4.2.

4.4.1 Design Check to Required Strengths Induced by the Seismic Base Shear

The required axial, flexural, and shear strengths are first extracted from the computer model, and then the column design strengths are hand-calculated.

Required strength for load combination LC1:

$M_u := 46.3 \quad \text{kip} - \text{ft} \qquad V_u := 7.6 \quad \text{kip} \qquad P_u := 255.4 \quad \text{kip}$

Trial section: Column_Size := "W14X74"

$E := 29000 \ \text{ksi} \qquad F_y := 50 \ \text{ksi} \qquad \phi_b := 0.9 \qquad L := 11.5 \cdot 12 \ \text{in}$

$A_g = 21.8 \ \text{in}^2 \qquad r_x = 6.04 \ \text{in} \qquad r_y = 2.48 \ \text{in} \qquad Z_x = 126 \ \text{in}^3$

$d = 14.17 \ \text{in} \qquad t_w = 0.45 \ \text{in}$

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 8.76 \ \text{ft} \qquad L_r = 27.9 \ \text{ft} \qquad M_r = 373 \ \text{kip} - \text{ft}$

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.5.1 (Seismic Provisions, Table I-8-1)

flange: $\lambda_{ps} := 0.3 \cdot \sqrt{\dfrac{E}{F_y}}$ $\quad \lambda_{ps} = 7.22 \quad \dfrac{b_f}{2 \cdot t_f} = 6.41 \quad \dfrac{b_f}{2 \cdot t_f} < \lambda_{ps}$ OK

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.26$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 55.83 \quad \frac{h}{t_w} = 25.3 \quad \frac{h}{t_w} < \lambda_{ps}$$ OK

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$l_x := L \qquad l_y := L \qquad k := 1.0$

$$\lambda_{c1} := \frac{k \cdot l_x}{\pi \cdot r_x} \cdot \sqrt{\frac{F_y}{E}} \quad \lambda_{c1} = 0.3 \qquad \lambda_{c2} := \frac{k \cdot l_y}{\pi \cdot r_y} \cdot \sqrt{\frac{F_y}{E}} \quad \lambda_{c2} = 0.74$$

$\lambda_c := \max(\lambda_{c1}, \lambda_{c2}) \quad \lambda_c = 0.74$

$F_{cr} := 0.658^{\lambda_c^2} \cdot F_y \qquad \lambda_c \le 1.5$ (E2-2)

$F_{cr} = 39.87$ ksi

$P_n := F_{cr} \cdot A_g$ (E2-1) $\qquad \phi_c := 0.85$ (E2-1)

$\phi_c \cdot P_n = 738.79$ kips

$\dfrac{P_u}{\phi_c \cdot P_n} = 0.35$

Bending Capacity (AISC LRFD section 16, Chapter F)

$L_b := 11.5 \qquad L_b = 11.5$ ft $\qquad L_p = 8.76$ ft

$C_b := 2.26$ (Cb is obtained from the computer program for the loading combination being considered)

$M_p := F_y \cdot \dfrac{Z_x}{12}$ (F1-1) $\qquad M_p = 525$ kip – ft

$$M_{n1} := C_b \cdot \left[M_p - (M_p - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \qquad L_p \le L_b \le L_r \qquad \text{(F1-2)}$$

$M_n := \min(M_p, M_{n1})$

$\phi_b \cdot M_n = 472.5$ kip – ft

$\dfrac{M_u}{\phi_b \cdot M_n} = 0.1$

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v := 0.9 \qquad A_w := d \cdot t_w$ (F.2.1)

$V_n := 0.6 \cdot F_y \cdot A_w \qquad \frac{h}{t_w} \le 2.45 \cdot \sqrt{\frac{E}{F_y}}$ (F2-1)

$\phi_v \cdot V_n = 172.17$ kips $\qquad \frac{V_u}{\phi_v \cdot V_n} = 0.04$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$R := \frac{P_u}{\phi_c \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_u}{\phi_b \cdot M_n} \right) \qquad \frac{P_u}{\phi_c \cdot P_n} \ge 0.2$ (H1-1a)

Demand Capacity Ratio: $R = 0.43$ W14x74 OK

4.4.2 Design Check to Required Axial Strengths Induced by Deformations at 2.0D_{bm}

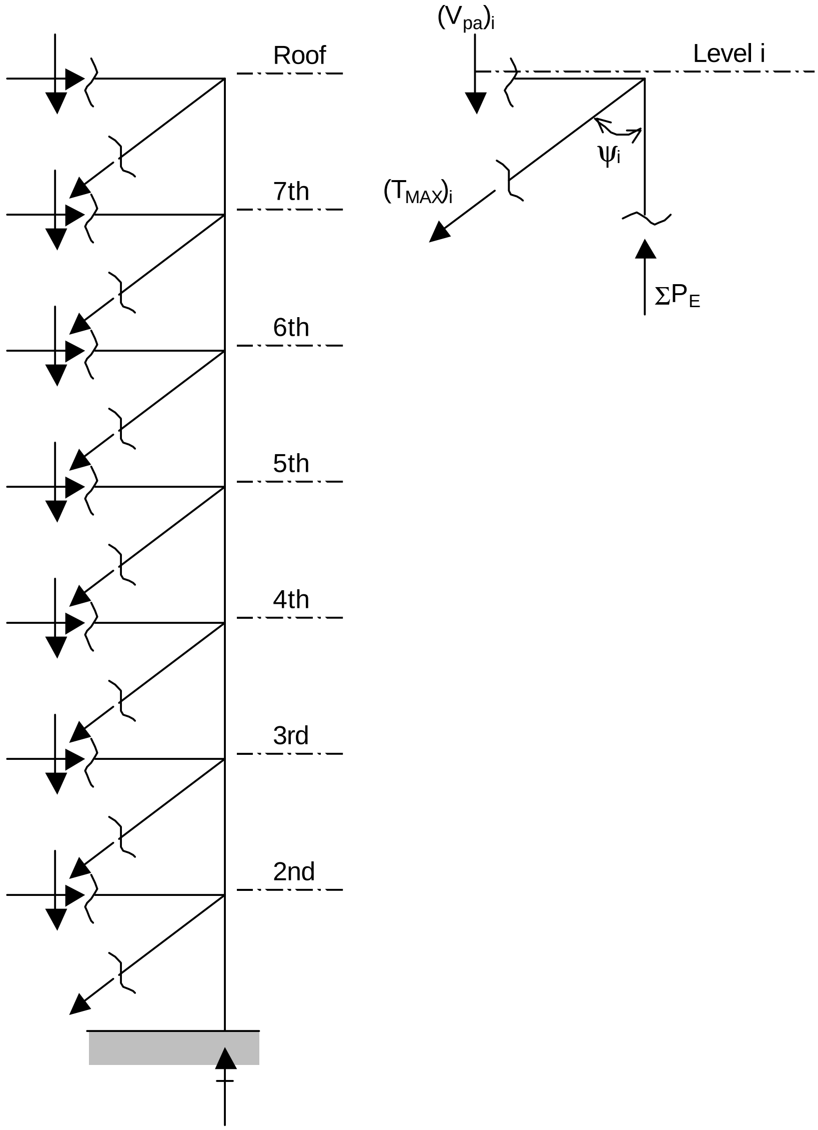


Figure 11. Axial Compression Demand on a Single-Diagonal BRBF Column

The required axial strength is defined in FEMA 450 section 8.6.3.5.3. Computation of the required axial strength is as shown in Figure 11 and Table 10. In computing Table10, the following were used.

- Beam size at all levels: W16x50 with M_p = 383 kip-ft and P_y = 735 kips
- Beam P_u is computed as described in section 4.3.2
- $M_{pa} = 1.18\left(1 - \frac{P_u}{P_y}\right)M_p$ for $\frac{P_u}{P_y} > 0.15$ (7.9), ASCE (1971)
- R_y = 1.1 for ASTM A992 (Seismic Provisions Table I-6-1)
- $L' = \text{clear beam distance} = 20'-(1'-3")-(2)(1'-10\,\frac{1}{2}") = 15'-0"$

  L' = center line dimension – column depth – 2 x gusset plate horizontal dimension

- $V_{pa} = \dfrac{2R_y M_{pa}}{L'}$
- $P_E = V_{pa} + T_{MAX} \cdot \cos?$
- Required compressive strength $P_u = 1.41\Sigma P_D + 0.5\Sigma P_L + \Sigma P_E$ (Eq. 9.5.2.7.1-1)
  See also section 3.3 for a description of load combination data.

Table 10. Column A/3 Required Axial Strengths at 2.0 D_{bm}

Column Below Level	Beam P_u (k)	Beam M_{pa} (ft-k)	Beam V_{pa} (k)	BRB T_{MAX}*cosψ (k)	Column ΣP_E (k)	Extracted from model ΣP_D (k)	Extracted from model ΣP_L (k)	Column P_u (k)
Roof	45	383	55	51	107	17	5	133
7th	117	380	55	84	245	39	17	308
6th	183	340	49	127	421	59	28	518
5th	244	302	44	155	619	80	39	751
4th	293	272	39	182	841	100	50	1007
3rd	329	250	36	196	1073	120	61	1272
2nd	340	243	35	237	1344	138	71	1574

$P_u := 518$ kip From Table 10

Trial section: Column_Size := "W14X74"

$E := 29000$ ksi $F_y := 50$ ksi $\phi_b := 0.9$ $L := 11.5 \cdot 12$ in $A_g = 21.8$ in^2

$r_x = 6.04$ in $r_y = 2.48$ in $d = 14.17$ in $t_w = 0.45$ in

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.5.1 (Seismic Provisions, Table I-8-1)

As described previously for frame beams, Pu is taken as the force corresponding to a deformation of 2.0 Δbm. This approach is chosen to meet the intent of the *Recommended Provisions* that permit flexural yielding of the frame columns but do not allow for compression instability at these high axial loads.

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.53$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 48.6 \qquad \frac{h}{t_w} = 25.3 \qquad \frac{h}{t_w} < \lambda_{ps} \quad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$\phi_c \cdot P_n = 739$ kips

$$\frac{P_u}{\phi_c \cdot P_n} = 0.7$$

Axial Compression Stability (AISC LRFD section 16, Chapter C)

A check seldom performed on frame columns but required by the spirit of the LRFD Specifications, *Seismic Provisions*, and *Recommended Provisions* will be performed here. Any braced-frame design on which the maximum axial required strength in frame columns is calculated through either (1) a load combination incorporating Ω_o, (2) a formal nonlinear analysis, or (3) a pseudo nonlinear analysis such as that on Table 10 meets the technical requirements of section C2 and should be checked accordingly. This is done to verify, as best as we can, the ability of the column section to withstand the formation of a hinge due to combined axial and flexural demands.

$$\frac{P_u}{0.85 \cdot \phi_c \cdot A_g \cdot F_y} = 0.66 < 1.0$$ (AISC LRFD Specifications section C.2.1a)

W14x74 OK

5. DESIGN OF INVERTED V-BRACED FRAME

5.1 Brace Demands and Brace Capacities

This section illustrates the design of the 2nd story BRB along line 6 between lines B and C. See Figure 6. First, the brace required strength is calculated utilizing the computer run results:

$\rho_Y = 1.23$
$P_E = 250.2$ kips (POSECCEQ$_Y$)
$P_D = 17.9$ kips
$P_L = 9.2$ kips
LC9: $P_u = 1.41\,P_D + 0.5\,P_L + \rho_Y\,P_E$
$P_u = 337.6$ kips

Then, the design strength is calculated in a similar fashion as shown in section 4.1.

$$\phi P_n = \phi P_{ysc} = \phi F_{ysc}\,A_{sc}$$
$$= 0.9 \text{ x } 38 \text{ ksi x } 10.5 \text{ in}^2 = 359.1 \text{ kips}$$

$$(\text{DCR}) = \frac{P_u}{f\,P_n} = \frac{337.6}{359.1} = 0.94 < 1.00 \qquad \text{OK}$$

5.2 Computation of 2.0D$_{bm}$, Brace Strains, and Adjusted Brace Strengths

With minor adjustments, the same procedure and equations described in section 4.2 are followed here in the creation of Tables 11 and 12. In calculating Δ_{bm} the effect of the vertical frame beam deflection is considered negligible. This assumption, which simplifies Δ_{bm} calculation, will be explicitly verified in section 5.3.3. Again after sizing the braces for strength, a brace manufacturer was given enough information to determine that for BF-2 BRBs the yield length can be approximated as one half of the work-point-to-work-point length, $L_{ysc} = 0.50L_1$. See Figure 8. Therefore,

Table 11. Strength Adjustment Factors for BF-2 BRBs

Story	A_{sc} (sq. in)	P_{bx} (k)	L_{ysc} (in)	Δ_{bx} (in)	Δ_{bm} (in)	$2.0\Delta_{bm}$ (in)	ε_{BRC} (%)	Adjustment Factors ω	$\omega\beta$	β
7th	3.00	66.0	113.4	0.09	0.43	0.86	0.76	1.15	1.18	1.02
6th	5.50	124.6	113.4	0.09	0.44	0.89	0.79	1.16	1.19	1.02
5th	7.00	169.1	113.4	0.09	0.47	0.94	0.84	1.18	1.20	1.02
4th	8.50	196.6	113.4	0.09	0.45	0.90	0.80	1.16	1.19	1.02
3rd	9.50	218.9	113.4	0.09	0.45	0.90	0.80	1.16	1.19	1.02
2nd	10.50	243.8	113.4	0.09	0.45	0.91	0.81	1.17	1.19	1.02
1st	11.00	240.8	123.1	0.09	0.46	0.93	0.76	1.15	1.18	1.02

Table 12. Adjusted BRB Strengths for BF-2 Frames

Story		Fy=46 ksi		
	A_{sc} (sq. in)	P_{ysc} (k)	T_{MAX} ωP_{ysc} (k)	C_{MAX} $\omega\beta P_{ysc}$ (k)
7th	3.00	138.00	159	162
6th	5.50	253.00	294	300
5th	7.00	322.00	378	388
4th	8.50	391.00	455	465
3rd	9.50	437.00	509	520
2nd	10.50	483.00	563	577
1st	11.00	506.00	583	595

5.3 Beam Design

This section will illustrate the design of the 2nd floor beam on line 6 between lines B and C. See Figure 6. The design of the BRBF beam is as previously described in section 4.3 with one addition. Because the BRBF configuration chosen is an inverted-V type, beams need to satisfy the additional requirements specified in FEMA 450 section 8.6.3.4. Compliance with these requirements is shown in section 5.3.3 of this *Steel TIPS* report.

5.3.1 Design Check to Required Strengths Induced by the Seismic Base Shear

The required axial, flexural, and shear strengths are first extracted from the computer model, and then the beam design strengths are hand-calculated.

Required strength for load combination LC9:

$M_u := 86$ kip – ft $\quad V_u := 14.2$ kip $\quad P_u := 225$ kip

Trial section: Beam_Size := "W16X50"

$E := 29000$ ksi $\quad F_y := 50$ ksi $\quad \phi_b := 0.9 \quad L := 360$ in

$A_g = 14.7$ in^2 $\quad r_x = 6.68$ in $\quad r_y = 1.59$ in $\quad Z_x = 92$ in^3

$d = 16.26$ in $\quad t_w = 0.38$ in

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 5.62$ ft $\quad L_r = 15.7$ ft $\quad M_r = 270$ kip – ft

<u>Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.6.1 (Seismic Provisions, Table I-8-1)</u>

flange: $\lambda_{ps} := 0.3 \cdot \sqrt{\dfrac{E}{F_y}} \qquad \lambda_{ps} = 7.22 \qquad \dfrac{b_f}{2 \cdot t_f} = 5.61 \qquad \dfrac{b_f}{2 \cdot t_f} < \lambda_{ps}$ OK

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.34$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \qquad \lambda_{ps} = 53.67 \qquad \frac{h}{t_w} = 37.4 \qquad \frac{h}{t_w} < \lambda_{ps} \quad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$l_x := \frac{L}{2}$ $\quad$ $l_y := \frac{L}{4}$ $\quad$ $k := 1.0$ $\quad$ ($k_y = 1.0$, $k_x < 1.0$. Use 1.0 as k_y governs)

$\lambda_{c1} := \frac{k \cdot l_x}{\pi \cdot r_x} \cdot \sqrt{\frac{F_y}{E}}$ $\quad$ $\lambda_{c1} = 0.36$ $\quad$ $\lambda_{c2} := \frac{k \cdot l_y}{\pi \cdot r_y} \cdot \sqrt{\frac{F_y}{E}}$ $\quad$ $\lambda_{c2} = 0.75$

$\lambda_c := \max(\lambda_{c1}, \lambda_{c2})$ $\quad$ $\lambda_c = 0.75$

$F_{cr} := 0.658^{\lambda_c^{\,2}} \cdot F_y$ $\quad$ $\lambda_c \le 1.5$ $\quad$ (E2-2)

$F_{cr} = 39.56$ ksi

$P_n := F_{cr} \cdot A_g$ (E2-1) $\quad$ $\phi_c := 0.85$ (E2-1)

$\phi_c \cdot P_n = 494.27$ kips $\quad$ $\frac{P_u}{\phi_c \cdot P_n} = 0.46$

Bending Capacity (AISC LRFD section 16, Chapter F)

Beam is braced at quarter points

$L_b := \frac{30}{4}$ $\quad$ $L_b = 7.5$ ft $\quad$ $L_p = 5.62$ ft

$C_b := 1.33$ $\quad$ (Cb is obtained from the computer program for the loading combination being considered)

$M_p := F_y \cdot \frac{Z_x}{12}$ (F1-1) $\quad$ $M_p = 383.33$ kip – ft

$M_{n1} := C_b \cdot \left[M_p - (M_p - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right]$ $\quad$ $L_p \le L_b \le L_r$ $\quad$ (F1-2)

$M_n := \min(M_p, M_{n1})$

$\phi_b \cdot M_n = 345$ kip – ft $\quad$ $\frac{M_u}{\phi_b \cdot M_n} = 0.25$

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v := 0.9$ $\qquad A_w := d \cdot t_w$ (F.2.1)

$V_n := 0.6 \cdot F_y \cdot A_w \qquad \frac{h}{t_w} \le 2.45 \cdot \sqrt{\frac{E}{F_y}}$ (F2-1)

$\phi_v \cdot V_n = 166.83$ kips $\qquad \frac{V_u}{\phi_v \cdot V_n} = 0.09$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$$R := \frac{P_u}{\phi_c \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_u}{\phi_b \cdot M_n} \right) \qquad \frac{P_u}{\phi_c \cdot P_n} \ge 0.2 \qquad \text{(H1-1a)}$$

Demand Capacity Ratio: $R = 0.68$ W16x50 OK

5.3.2 Design Check to Required Axial Strengths Induced by Deformations at 2.0D_{bm}

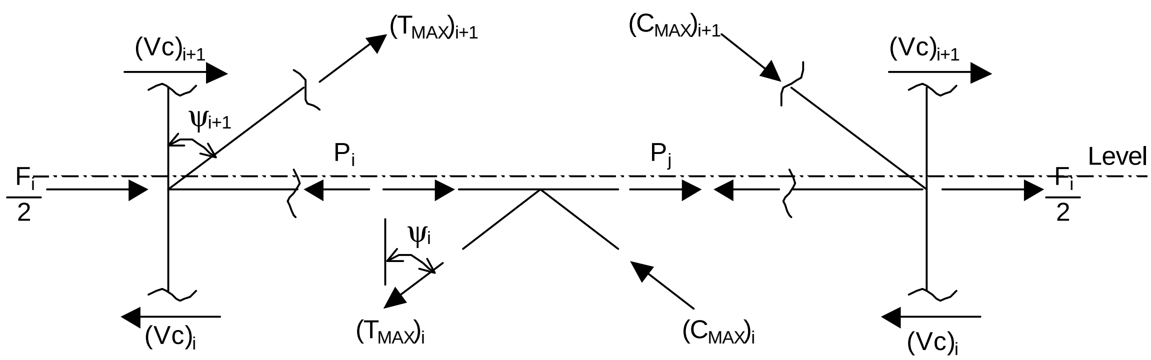


Figure 12. Required Axial Strengths of Second-Floor Beam per FEMA 450 Section 8.6.3.4.1.1

Since only an elastic analysis is performed, certain assumptions must be made to compute the axial force in the frame beam. These produce conservative results. They are:

- $(Vc)_{i+1}$=(Vc)=0 (Shears in columns are assumed to be zero)
- Fi is the sum of story collector forces corresponding to the mechanism under consideration. Collector forces are assumed equal at each end of the frame.

Alternatively, nonlinear analyses may be performed from which the actual demands in the members can be extracted.
Then,

$$F_i := TMAX_i \cdot \sin(\psi_i) + CMAX_i \cdot \sin(\psi_i) - TMAX_{i+1} \sin(\psi_{i+1}) - CMAX_{i+1} \cdot \sin(\psi_{i+1})$$

$$P_i := TMAX_{i+1} \cdot \sin(\psi_{i+1}) + \frac{F_i}{2}$$

$$P_j := P_i - TMAX_i \cdot \sin(\psi_i) - CMAX_i \cdot \sin(\psi_i)$$

$$P_u := \max(P_i, P_j)$$

For this example, level "i"=2nd floor. From Table 12:

$TMAX_{i+1} := 563$ k $\quad CMAX_{i+1} := 577$ k $\quad \psi_{i+1} := 52.52$ deg (2nd story)

$TMAX_i := 583$ k $\quad CMAX_i := 595$ k $\quad \psi_i := 46.98$ deg (1st story)

And,

$P_u = 425$ kips

Mu and Vu were obtained from the computer model due to factored vertical loads: 1.41D + 0.5L.

$M_u := 38.0$ kip – ft $\quad V_u := 10.6$ kips $\quad P_u = 425$ kips

Trial section: Beam_Size := "W16X50"

$E := 29000$ ksi $\quad F_y := 50$ ksi $\quad \phi_b := 0.9 \quad L := 30 \cdot 12$ in

$A_g = 14.7$ in^2 $\quad r_x = 6.68$ in $\quad r_y = 1.59$ in $\quad Z_x = 92$ in^3

$d = 16.26$ in $\quad t_w = 0.38$ in

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 5.62$ ft $\quad L_r = 15.7$ ft $\quad M_r = 270$ kip – ft

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.6.1 (Seismic Provisions, Table I-8-1)

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.64$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 45.51 \qquad \frac{h}{t_w} = 37.4 \qquad \frac{h}{t_w} < \lambda_{ps} \qquad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$\phi_c \cdot P_n = 494.27$ kips

$$\frac{P_u}{\phi_c \cdot P_n} = 0.86$$

Bending Capacity (AISC LRFD section 16, Chapter F)

Beams are braced at quarter points

$\phi_b \cdot M_n = 345$ kip – ft

$$\frac{M_u}{\phi_b \cdot M_n} = 0.11$$

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v \cdot V_n = 166.83$ kips

$$\frac{V_u}{\phi_v \cdot V_n} = 0.06$$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$$R := \frac{P_u}{\phi_c \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_u}{\phi_b \cdot M_n} \right) \qquad \frac{P_u}{\phi_c \cdot P_n} \geq 0.2 \qquad \text{(H1-1a)}$$

Demand Capacity Ratio: $R = 0.96$ W16x50 OK

5.3.3 Design Checks Specific to Inverted-V BRBFs

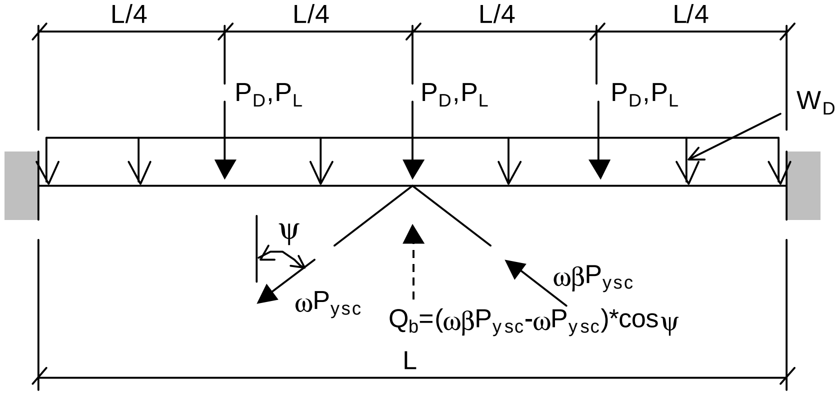


Figure 13. Applied Loads on Beam from Adjusted Brace Strengths.

Trial section: Beam_Size := "W16X50"

$E := 29000$ ksi $F_y := 50$ ksi $L := 360$ in $I_x := 659$ in^4 $S_x := 81.0$ in^3

$P_D := 9.56$ kips $W_D := 0.191\ \frac{k}{ft}$ $P_L := 5.63$ kips

$\omega\beta P_{ysc} := 595$ kips $\omega P_{ysc} := 583$ kips $\psi := 46.98$ deg

Compute maximum negative moments at beam end supports and maximum deflection at midspan assuming braces are not present. Beam is fixed at ends.

Effects due to P D:

$$M_{D1} := \frac{5}{16}\cdot P_D\cdot\left(\frac{L}{12}\right) \qquad M_{D1} = 89.63 \text{ kip}-\text{ft} \qquad \Delta_{D1} := \frac{1}{96}\cdot\frac{P_D\cdot L^3}{E\cdot I_x} \qquad \Delta_{D1} = 0.24 \text{ in}$$

Effects due to W D:

$$M_{D2} := \frac{1}{12}\cdot W_D\cdot\left(\frac{L}{12}\right)^2 \qquad M_{D2} = 14.32 \text{ kip}-\text{ft} \qquad \Delta_{D2} := \frac{1}{384}\cdot\frac{\left(\frac{W_D}{12}\cdot L^4\right)}{E\cdot I_x} \qquad \Delta_{D2} = 0.04 \text{ in}$$

Effects due to P L:

$$M_L := \frac{5}{16}\cdot P_L\cdot\left(\frac{L}{12}\right) \qquad M_L = 52.78 \text{ kip}-\text{ft} \qquad \Delta_L := \frac{1}{96}\cdot\frac{P_L\cdot L^3}{E\cdot I_x} \qquad \Delta_L = 0.14 \text{ in}$$

Effects due to Q b:

$$Q_b := \left(\omega\beta P_{ysc} - \omega P_{ysc}\right)\cdot\cos\left(\psi\cdot\frac{\pi}{180}\right) \qquad Q_b = 8.19 \text{ kips}$$

$$M_E := \frac{-1}{8}\cdot Q_b\cdot\left(\frac{L}{12}\right) \qquad M_E = -30.7 \text{kip}-\text{ft} \qquad \Delta_E := \frac{-1}{192}\cdot\frac{Q_b\cdot L^3}{E\cdot I_x} \qquad \Delta_E = -0.1 \text{ in}$$

Strength check (FEMA 450 section 8.6.3.4.1.1):

$M_y := F_y \cdot \frac{S_x}{12}$ $\qquad M_y = 337.5 \quad kip - ft$

U = 1.41D + 0.5L + E

$M_u := 1.41 M_{D1} + 1.41 M_{D2} + 0.5 M_L + M_E \qquad M_u = 142.26 \quad kip - ft$

$\frac{M_u}{M_y} = 0.42$ $\qquad$ < 1.00 OK

Stiffness check (FEMA 450 section 8.6.3.4.1.2):

$\Delta_{MIDDLE} := \Delta_{D1} + \Delta_{D2} + \Delta_E \qquad \Delta_{MIDDLE} = 0.175 \quad in$ (downward) =L/2,057 OK

W16x50 OK

As illustrated in the previous calculations, the beam contribution to brace deformation is negligible. This finding validates the assumption made in section 5.2 and allows the use of the Δ_{bm} values shown in Table 11 in developing the displacement protocol while complying with FEMA 450 section 8.6.3.4.1.3. The results of the previous section also allow it to be stated that compliance with FEMA 450 section 8.6.3.4.1.2 for other possible load combinations is achieved by inspection and that there is no need to perform further calculations.

5.4 Column Design

This section illustrates the design of column C/6 between the 1st and 2nd levels. See Figure 6. The same procedure outlined in section 4.4 will be followed here.

5.4.1 Design Check to Required Strengths Induced by the Seismic Base Shear

The required axial, flexural, and shear strengths are first extracted from the computer model, and then the column design strengths are hand-calculated.

Required strength from computer model for load combination LC9:

$M_u := 234.5$ kip − ft $\quad V_u := 20.6$ kip $\quad P_u := 1045$ kip

Trial section: Column_Size := "W14X211"

$E := 29000$ ksi $\quad F_y := 50$ ksi $\quad \phi_b := 0.9 \quad L := 14 \cdot 12$ in

$A_g = 62$ in^2 $\quad r_x = 6.55$ in $\quad r_y = 4.07$ in $\quad Z_x = 390$ in^3

$d = 15.72$ in $\quad t_w = 0.98$ in

The following values are from LRFD 3rd Ed., Table 5-3:

$L_p = 14.4$ ft $\quad L_r = 76$ ft $\quad M_r = 1122$ kip − ft

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.5 (Seismic Provisions, Table I-8-1)

flange: $\lambda_{ps} := 0.3 \cdot \sqrt{\dfrac{E}{F_y}} \quad \lambda_{ps} = 7.22 \quad \dfrac{b_f}{2 \cdot t_f} = 5.06 \quad \dfrac{b_f}{2 \cdot t_f} < \lambda_{ps}$ OK

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.37$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 52.74 \quad \frac{h}{t_w} = 11.6 \quad \frac{h}{t_w} < \lambda_{ps} \quad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$l_x := L$ $\quad l_y := L$ $\quad k := 1.0$

$\lambda_{c1} := \frac{k \cdot l_x}{\pi \cdot r_x} \cdot \sqrt{\frac{F_y}{E}}$ $\quad \lambda_{c1} = 0.34$ $\quad \lambda_{c2} := \frac{k \cdot l_y}{\pi \cdot r_y} \cdot \sqrt{\frac{F_y}{E}}$ $\quad \lambda_{c2} = 0.55$

$\lambda_c := \max(\lambda_{c1}, \lambda_{c2})$ $\quad \lambda_c = 0.55$

$F_{cr} := 0.658^{\lambda_c^2} \cdot F_y$ $\quad \lambda_c \le 1.5$ $\quad$ (E2-2)

$F_{cr} = 44.14$ ksi

$P_n := F_{cr} \cdot A_g$ (E2-1) $\quad \phi_c := 0.85$ (E2-1)

$\phi_c \cdot P_n = 2326$ kips

$$\frac{P_u}{\phi_c \cdot P_n} = 0.45$$

Bending Capacity (AISC LRFD section 16, Chapter F)

$L_b := 14$ $\quad L_b = 14$ ft $\quad L_p = 14.4$ ft

$C_b := 1.95$ (Cb is obtained from the computer program for the loading combination being considered)

$M_p := F_y \cdot \frac{Z_x}{12}$ (F1-1) $\quad M_p = 1625$ kip – ft

$$M_{n1} := C_b \cdot \left[M_p - (M_p - M_r) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \quad L_p \le L_b \le L_r \qquad \text{(F1-2)}$$

$M_n := \min(M_p, M_{n1})$

$\phi_b \cdot M_n = 1462.5$ kip – ft

$$\frac{M_u}{\phi_b \cdot M_n} = 0.16$$

.

Shear Capacity (AISC LRFD section 16, Chapter F)

$\phi_v := 0.9 \qquad A_w := d \cdot t_w$ (F.2.1)

$V_n := 0.6 \cdot F_y \cdot A_w \qquad \frac{h}{t_w} \le 2.45 \cdot \sqrt{\frac{E}{F_y}}$ (F2-1)

$\phi_v \cdot V_n = 415.95$ kips

$\frac{V_u}{\phi_v \cdot V_n} = 0.05$

Bending-Axial Interaction (AISC LRFD section 16, Chapter H)

$R := \frac{P_u}{\phi_c \cdot P_n} + \frac{8}{9} \cdot \left(\frac{M_u}{\phi_b \cdot M_n} \right) \qquad \frac{P_u}{\phi_c \cdot P_n} \ge 0.2$ (H1-1a)

Demand Capacity Ratio: $R = 0.59$ W14x211 OK

5.4.2 Design Check to Required Axial Strengths Induced by Deformations at 2.0Δ_{bm}

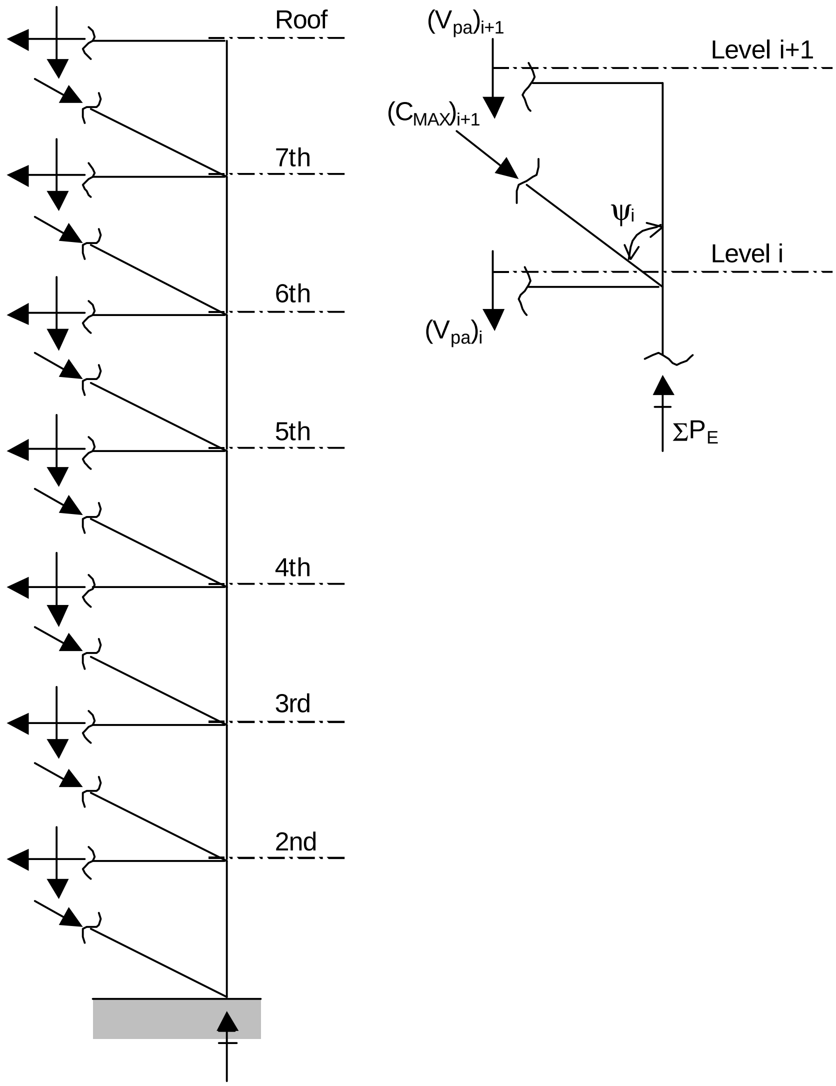


Figure 14. Axial Compression Demand on Inverted-V BRBF Column

The required axial strength is defined in FEMA 450 section 8.6.3.5.3. Computation of the required axial strength is as shown in Figure 14 and Table 13. Computation of Table 13 follows the same procedure described in section 4.4.2 with L' = 25'- 0".

Table 13. Column C/6 Required Axial Strengths at 2.0 D_{bm}

Column Below Level	Beam P_u (k)	Beam M_{pa} (ft-k)	Beam V_{pa} (k)	BRB C_{MAX}*cosψ (k)	Column ΣP_E (k)	Extracted from model ΣP_D (k)	Extracted from model ΣP_L (k)	Column P_u (k)
Roof	127	374	33	99	33	13	4	52
7th	234	308	27	183	158	37	14	217
6th	302	267	23	236	364	63	28	467
5th	361	230	20	283	620	90	42	768
4th	404	204	18	316	921	116	56	1113
3rd	448	177	15	351	1252	143	70	1489
2nd	425	191	17	406	1620	169	84	1900

$P_u := 1900$ kip From Table 13

Trial section: Column_Size := "W14X211"

$E := 29000$ ksi $F_y := 50$ ksi $\phi_b := 0.9$ $L := 14 \cdot 12$ in $A_g = 62$ in^2

$r_x = 6.55$ in $r_y = 4.07$ in $d = 15.72$ in $t_w = 0.98$ in

Width-Thickness Ratios. Comply with FEMA 450 section 8.6.3.5.1 (Seismic Provisions, Table I-8-1)

As described previously for frame beams, Pu is taken as the force corresponding to a deformation of 2.0 Δbm.

web: $\dfrac{P_u}{\phi_b \cdot A_g \cdot F_y} = 0.68$

$$\lambda_{ps} := 1.12 \cdot \sqrt{\frac{E}{F_y}} \cdot \left(2.33 - \frac{P_u}{\phi_b \cdot F_y \cdot A_g}\right) \quad \lambda_{ps} = 44.48 \quad \frac{h}{t_w} = 11.6 \quad \frac{h}{t_w} < \lambda_{ps} \quad \text{OK}$$

Section is seismically compact

Axial Compression Capacity (AISC LRFD section 16, Chapter E)

$\phi_c \cdot P_n = 2326$ kips

$$\frac{P_u}{\phi_c \cdot P_n} = 0.82$$

Axial Compression Stability (AISC LRFD section 16, Chapter C)

$$\frac{P_u}{0.85 \cdot \phi_c \cdot A_g \cdot F_y} = 0.85 \quad < 1.0$$

(AISC LRFD Specifications section C.2.1a)

W14x211 OK

6. DEVELOPMENT OF A DISPLACEMENT PROTOCOL FOR TESTING

This section illustrates the development of a displacement protocol for testing. Testing may be project specific with a protocol developed from calculated brace displacements (as done in this section), or it may be generic, performed by brace manufacturers to qualify their braces for a range of applications. It will be here assumed that existing test results are such that they do not meet FEMA 450 section 8.6.3.2.2, the applicability article, when compared to this example's conditions and therefore project-specific testing is required. It is worth noting that brace deformations are but one of the variables determining applicability of test results to an actual building design. Other variables include interstory drift ratios, member sizes, brace angles, brace-end connections, and so on. These need to be addressed in establishing compliance with the applicability article.

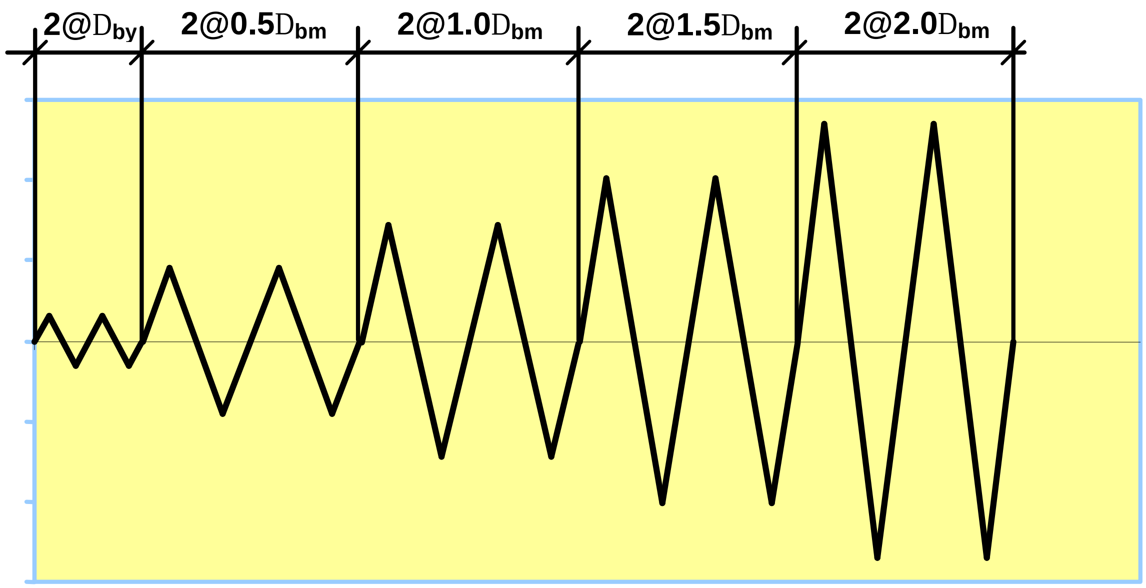


Figure 15. BRB Displacement Protocol

In cases in which a reasonable match between available test results and project conditions is not possible, the applicability article allows for calculation of magnitude and distribution of internal strains to justify the use of available test results. Such strain calculations need to be approved by the project peer reviewer or building official for the extrapolation to be valid. For this example, it will be assumed that extrapolations are not technically justified and testing is therefore required.

The displacement protocols defined in this section are the protocols being proposed for the 2005 edition of the *Seismic Provisions*. Two test types, with two different displacement protocols, are required: a uniaxial test and a subassemblage test. For both histories, the graphic representation of the displacement protocol is shown in Figure 15. For both tests the properties of the brace specimens should match as closely as practicable the values of brace strength (P_{ysc}), and maximum strain (ε_{BRC}). Additionally, it is also advisable to match brace angle (ψ), brace length, and design drift ratio?(θ_M)? For diagrammatic representations of possible test setups refer to FEMA 450 Figures C8.6.3.5 and C8.6.3.6. The quantity Δ_b represents both the total BRB axial deformation for the uniaxial test and the total BRB end rotation for the subassemblage test.

6.1 Uniaxial Test (Specimen 1)

The 6th story BRB of BF-1 frames will be chosen for this test. This is the brace described and designed in section 4.1. See Figure 6. This brace is chosen so that, for a single-diagonal BRBF, all aspects of BRB design are illustrated; namely, sizing for strength, computing Δ_{bm}, computing BRB strains and adjusted BRB strengths, and developing a displacement protocol. In an actual building project the largest brace size would normally be chosen. Following are the required test parameters.

- $P_{ysc} \geq 138\ \text{kips}$ (see Table 9)
- $\text{BRB length} \approx 23'\text{–}0''$ (inferred from Table 8)
- $e_{BRC} \geq 0.98\%$ (see Table 8)

axial deformation $\Delta_{by} = \dfrac{F_{ysc} L_{ysc}}{E}$

where F_{ysc} = 46 ksi, L_{ysc} is as shown in Table 8 and E = 29000 ksi

- axial deformation $\Delta_{by} = 0.29''$
- axial deformation $\Delta_{bx} = 0.18''$ (see Table 8)
- axial deformation $2.0\Delta_{bm} = 1.79''\ (\approx 6.17\Delta_{by})$ (see Table 8)

The testing protocol is as described in Table 14.

Table 14. Example Uniaxial Testing Protocol

Cycles	Inelastic Deformation		
	Per cycles	Total	Cumulative
2 @ Δ_{by}	2 x 4 @ 0 =	0	0
2 @ $0.5\Delta_{bm}$	2 x 4 @ 0.54 =	4.3	4.3
2 @ $1.0\Delta_{bm}$	2 x 4 @ 2.09 =	16.7	21.0
2 @ $1.5\Delta_{bm}$	2 x 4 @ 3.63 =	29.0	50.1
2 @ $2.0\Delta_{bm}$	2 x 4 @ 5.17 =	41.4	91.4
8 @ $1.5\Delta_{bm}$*	8 x 4 @ 3.63 =	116.2	207.8

* As required by the proposed 2005 edition of the *Seismic Provisions*, eight additional cycles @ $1.5\Delta_{bm}$ were added to reach a cumulative inelastic deformation of 200.

6.2 Subassemblage Test (Specimen 2)

The 2nd story BRB, second to largest brace size, of BF-2 frames will be chosen for this test. This is the brace described and designed in section 5.1. See Figure 6. This brace is chosen so that, for an inverted-V BRBF, all aspects of BRB design are illustrated; namely, sizing for strength, computing Δ_{bm}, computing BRB strains and adjusted BRB strengths, and developing a displacement protocol. In an actual building project the largest brace size that an experimental facility can test in a subassemblage mode would normally be chosen. This may not be the largest brace size in the project.

As described previously, Δ_b is the total BRB end rotation. To compute Δ_{bm}, Δ_{bx} was extracted from the applicable load combination causing the largest drift. Within a bay, the BRB with the largest end rotations was chosen. The rotations at both the bottom and top ends of the BRB were determined. For this example, at the bottom of the governing BRB the Δ_{bx} rotation is 0.00212 radians and at the top of the

BRB is 0.000268 radians. With Δ_{bx} = 0.00212 radians the following test parameters were computed and are summarized in Table 15.

- $P_{ysc} \geq 483$ kips (see Table 12)
- BRB length $\approx$ 20'–6" (inferred from Table 11)
- $e_{BRC} \geq 0.81\%$ (see Table 12)
- brace end rotation $\Delta_{bx} = 0.00212$ radians
- brace end rotation $\Delta_{by} = 0.00212 \text{ radians x } \frac{P_{ysc}}{P_{bx}} = 0.00420$ radians
- brace end rotation $2.0\Delta_{bm} = 0.0212$ radians ($\approx 5.0\Delta_{by}$)

On Figure C8.6.3.6, from top to bottom, the second subassemblage configuration is used here to illustrate the displacement protocol. A constant brace end rotation of $2.0\Delta_{bm}$ will be imposed on one end of the brace while reversing, increasing axial loads are applied. The maximum axial loads applied are the adjusted BRB strengths, T_{MAX} and C_{MAX}. See Table 12. This approach is chosen to achieve a hysteresis loop that can be readily compared to one obtained from a uniaxial test. Intermediate axial load values are interpolated between P_{ysc} and T_{MAX} or C_{MAX} depending on the testing cycle. Another approach for subassemblage testing is to hold the axial load constant and apply reversing, increasing brace end rotations. However, such approach doesn't appear as straightforward and is not followed here.

Table 15. Example Subassemblage Testing Protocol

Cycles	Total end rotation (rad)	Applied Axial Load	
		Tension (k)	Compression (k)
2 @ Δ_{by}	0.0212	483	483
2 @ $0.5\Delta_{bm}$	0.0212	503	507
2 @ $1.0\Delta_{bm}$	0.0212	523	530
2 @ $1.5\Delta_{bm}$	0.0212	543	554
2 @ $2.0\Delta_{bm}$	0.0212	563	577

7. GUSSET-PLATE BEHAVIOR AND DESIGN

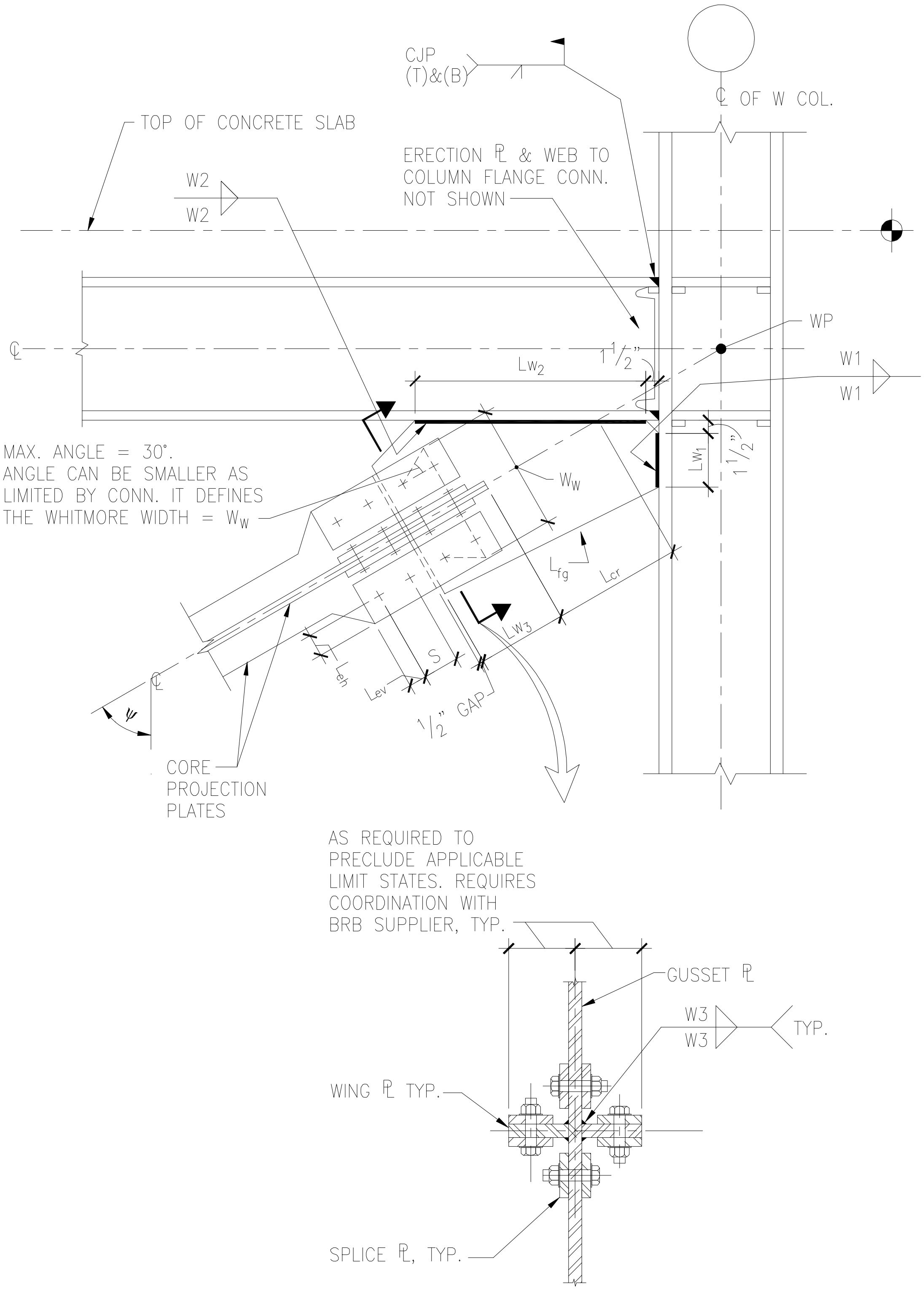


Figure 16. BRB-Beam-Column Connection

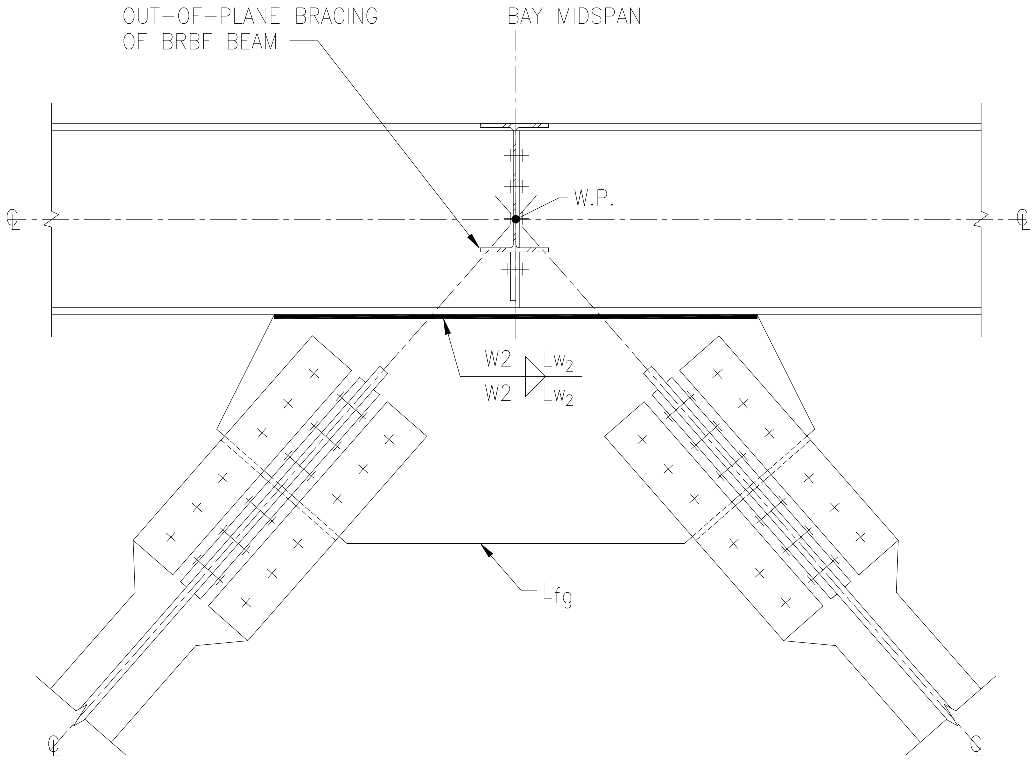


Figure 17. BRB-Beam Connection

Figures 16 and 17 illustrate typical BRB-to-beam-column and BRB-to-beam connections. Because of the high axial forces transferred through the connections and the large interstory drifts associated with θ_M, these connections will withstand strength and deformation demands requiring ductile detailing.

At the beam-to-column connection shown in Figure 16, the structural engineer is advised to utilize a connection with experimentally proven plastic rotation capacity. This is especially important for single-diagonal BRBFs. From Table 6 we see that the maximum design drift ratio $(\theta_M)_{max}$ = 1.72%. Since beam-to-column connections start yielding at design drift ratios of about 1%, deformation demands in excess of the connection's yield capacity can be expected.

The design of the gusset itself is as important to the adequate performance of BRBFs (and all concentrically braced frame systems) as is the correct design of BRBs, beams, columns, and beam-to-column connections. As of the writing of this *Steel TIPS* report, early 2004, however, the state of the practice seems generally not to benefit from existing information regarding gusset-plate behavior and design. Structural engineers are not inclined to use as resources the writings of Richard (1986), Gross (1990), Thornton (1991), and Astaneh-Asl (1998). The structural engineering profession still produces gusset-plate designs that appear too large and expensive and with limited ductility capacity. As a result, the concentric braced-frame system (BRBF, SCBF, and Ordinary Concentric Braced Frames [OCBF]) may not have the ductility to reach the drift required during severe ground motions. If the concentric braced-frame system's connections do not have the capacity to sustain the deformations that it will experience, then the system does not possess as much ductility as is assumed in the R value.

A step-by-step example of how to detail a BRBF gusset connection will not be shown here. Enough information is illustrated in the references cited for a structural engineer to arrive at a ductile detail. In the authors' opinion, the reason that some gusset-plate designs do not exhibit ductile behavior is that the guidelines in the references are not followed, not because of lack of guidelines. That is why this section encourages readers to use the references cited and will only try to complement the references by highlighting certain design aspects.

For the connections shown in Figures 16 and 17, the following design guidelines are offered to ensure acceptable gusset-plate behavior.

- Use of the Uniform Force Method (AISC, 2001) provides more compact gusset plates and less expensive designs.
- Use of an alternate work point such as that illustrated by Sabelli in his design example (SEAOC, 2003) produces even more compact gussets, which are desirable.
- Gusset width considered in the analysis need not correspond to an angle of 30 degrees (the "Whitmore" width) but may be smaller if the applicable limit states are precluded.
- The length of the wing plate, L_{w3}, may extend past the splice plates to provide for adequate load transfer from the wing plates and minimize the buckling length, L_{cr}.
- Check appropriate limit states (See AISC LRFD 3rd Ed. Specifications Chapter K) at the gusset-to-beam and gusset-to-column interfaces and avoid adding stiffeners or doubler plates within the beam or column if not required.
- Check the length of the free edge of the gusset, L_{fg}. The length of the free edge of the gusset may become too long and buckle under the rotations of the beam-column joint. This phenomenon has been described analytically by Richard (1986) and witnessed experimentally by Gross (1990), López et al., (2002), and Tsai et al. (2003b). All the variables contributing to buckling of the free edge are not yet completely understood nor are its effects, or lack of, on adequate gusset-plate behavior. Until publication of new research results, use of proposed equation (2.3) in Astaneh-Asl (1998) to check L_{fg} is encouraged.
- A gusset plate connected to both the beam and column flanges functions as a haunch in a fully restrained beam-column connection. In addition to the forces transmitted from the braces to the framing members, the kinematics of frame deformation imposes forces transverse to the brace axis on gussets. This aspect of gusset-plate behavior has not been adequately studied, and the effects, both detrimental and beneficial, of these transverse forces are not well understood. Specific design procedures and appropriate details have not been established. Stress concentrations resulting from this haunch-type behavior should be considered in the detailing of gusset plates to ensure that weld fracture does not limit the system performance. In addition, the effects of having such a haunch in the frame should be addressed in the analysis by modeling the restraint at these connections. The resulting flexural forces should be used in the design of beams and columns.

8. SAMPLE SPECIFICATION

The following sample specification is presented as a possible guideline. Blanks are provided so that the specifics of an actual building project can be inserted. The structural engineer of record is to review and edit the sample specification to suit the project needs.

BUCKLING-RESTRAINED BRACES

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes:

1. Furnishing Buckling Restrained Braces (BRBs).

2. Engineering design of Buckling Restrained Braces.

3. Qualification of BRBs by uniaxial and subassemblage cyclic testing.

1.2 REFERENCES

A. Standards listed below apply where designation is cited in this Section. Where the applicable year of adoption or revision is not listed below, the latest edition applies.

B. AISC - American Institute of Steel Construction

1. Specification - LRFD Specification for Structural Steel Buildings, 1999 Edition.

C. FEMA – Federal Emergency Management Agency

1. 450 -*2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*.

D. ASTM - American Society for Testing and Materials

1. A6 - Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use.

2. A36 - Specification for Steel.

3. A500 - Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing.

4. A572 - Specification for Steel

E. AWS - American Welding Society

1. Structural Welding Code – Steel AWS D1.1.

F. JIS - Japanese Industrial Standard

1. G 3136 SN400 B - Rolled Steels for Building Structure.

2. G 3466 STKR 400 - Carbon Steel Square Pipes for General Structural Purposes.

1.3 DEFINITIONS

A. Buckling Restrained Brace (BRB): Specialty structural brace element consisting of an axial force resisting core of steel plates encased by a system that prevents buckling of the steel core.

1.4 SUBMITTALS

A. Submit the following in accordance with Section _____. All requested submittals shall be furnished in English language.

B. Within __ days of award of contract, furnish Qualification Testing Report evidencing manufacturer's compliance with Article 2.1 D.

1. The Qualification Testing Report shall conform to requirements of Section 8.6.3.7.9 of FEMA 450.

2. If project specific testing is required to supplement available test data, include schedule for fabrication of BRB test specimens, description of proposed testing program and name of test facility and schedule for testing and reporting.

C. Manufacturer's Quality Assurance Plan: Conform to requirements of Article 1.5A, "Quality Assurance".

1. An authorized representative of the manufacturer shall certify the validity of the Plan by signing and dating.

D. Engineering Design: Refer to Article 2.1A for design requirements.

1. Design Drawings: Show size and configuration of steel core for full length of BRB. Indicate casing size, thickness and length.

2. Calculations: Provide design calculations showing the adequacy of proposed BRBs to achieve Performance Requirements specified herein.

3. Certification: In accordance with Article 2.1A, Design Requirements.

4. Submit final drawings, calculations and certifications that include the final dimensions of steel core plates based on results of coupon testing of steel to be employed in Work.

a. The Design Engineer shall seal final design drawings, calculations and required certification.

b. Submittal shall be accompanied by the results of coupon testing.

E. Shop Drawings:

1. Show location and size of BRBs. Give complete information necessary for fabrication of elements of structural steel frame to receive braces and fabrication of connection plates. Show methods of assembly, including type and size of connectors, hole diameter, and preparation and finish of faying surfaces. Identify tolerances for fabrication and erection.

2. Provide final core plate dimensions based on results of coupon testing of steel.

F. Certificates of compliance with specified standards:

1. Steel.

2. Welding filler materials.

G. Certified material test reports: Submit to Testing Laboratory for record purposes.

1. All steel: Tensile tests and chemical analysis. Include trace elements for steel core plates.

2. Steel Core Plates:

a. Coupon test results for each lot of steel used in fabrication showing initial yield, ultimate tensile stress, and ultimate elongation.

b. Charpy V-Notch testing for plates 2 inches (50 mm) and thicker.

c. Welding electrodes: Include tensile, elongation, and CVN toughness tests. Identify diffusible hydrogen.

H. Welder Performance Qualification Records (WPQR's).

I. Written Welding Procedure Specification (WPS) in accordance with AWS D1.1 requirements for each different welded joint proposed for use, whether prequalified or qualified by testing.

1. Indicate as-detailed configuration.

2. Identify specific filler material and manufacturer.

J. Procedure Qualification Record (PQR) in accordance with AWS D1.1 for all procedures qualified by testing.

K. Submit Quality Assurance test and inspection reports to Testing Laboratory for record purposes.

1.5 QUALITY ASSURANCE

A. Manufacturer Qualifications: Shall have manufactured and successfully tested braces in accordance with Article 2.1D, "Qualification Tests" prior to opening of bids.

B. Design Engineer Qualifications: Civil Engineer, registered in the state where the project is located that is knowledgeable with the results of cyclic testing of BRBs and experienced in the design of BRBs based on engineering analysis.

C. Quality Assurance Plan: The manufacturer shall have a detailed Quality Assurance plan that shall include descriptions of manufacturing procedures, quality control testing program for materials, and all points of internal inspection and sign-off for control and monitoring of the fabrication and assembly process. Plan shall include BRB-manufacturer furnished Quality Assurance for erection.

1. Plan shall include attendance at pre-erection conference by Manufacturer's Representative and a minimum of one visit thereafter to observe installation of braces.

D. Qualification Testing: Refer to Article 2.1D for requirements.

E. Pre-Erection Conference: Contractor shall schedule meeting with Architect, BRB-manufacturer, and the steel erector's personnel supervising installation of buckling restrained braces to review installation procedures including handling, fit-up and fastening.

1.6 SCHEDULING

A. Furnish schedule for Buckling Restrained Brace manufacture and delivery within __ days of award of contract.

1.7 DELIVERY, STORAGE AND HANDLING

A. Manufacturer to provide protection for the braces to ensure against damage during shipping.

B. Contractor to provide proper lay-down and storage areas. Manufacturer to coordinate with the Contractor on delivery dates.

PART 2 - PRODUCTS

2.1 DESIGN AND PERFORMANCE REQUIREMENTS

A. Design Requirements:

1. Engage a Civil Engineer, licensed in the state where the project is located, to design braces to achieve the Performance Criteria. Design shall be based on detailed examination and understanding of the results of qualifying cyclic tests and interpolation or extrapolation of results to project conditions.

2. Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains that are consistent with or less severe than the tested assemblies and that considers the adverse effects of larger material and variations in material properties.

3. For stability calculations, beams, columns and gussets adjoining the brace shall be considered.

4. Consider the effect of end rotations corresponding to the Design Story Drift.

B. Performance Criteria:

1. Yield strength of steel core shall be as indicated on the Drawings to within the tolerances specified. Proportion steel cores to satisfy requirement within specified tolerances using coupon test data for steel furnished for project.

2. The portion of the steel core that projects beyond the casing shall be designed to develop (155% will satisfy this example's requirements)% of the initial yield force of the BRB without initiation of fracture.

3. Braces shall provide for stable cyclic displacement (lengthening and shortening) corresponding to the required deformation capacities indicated on Drawings.

   a. Hysteretic behavior in the non-linear range shall show no sign of degradation or loss of strength.

   b. Graphs of test results shall show no signs of pinched hysteretic behavior.

4. Tension and compression shall be resisted entirely by the steel core. The buckling restraining system shall limit local and overall buckling of the brace without restraining the steel core from transverse expansion and longitudinal shortening for the required deformation capacities indicated on Drawings.

C. Coupon Tests: Perform coupon test results for each lot of steel used in fabrication of steel cores showing initial yield, ultimate tensile stress, and ultimate elongation. Coupons shall be taken from plates at point of brace manufacture and shall be used as the basis for brace design.

D. Qualification Tests: The design of braces shall be based on results from qualifying cyclic tests. Tests shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotation demands and the other may be either a uniaxial or subassemblage test.

1. Qualification Tests shall conform to requirements of Section 8.6.3.7 of FEMA 450.

2. Qualification tests are permitted to be based on documented full-scale cyclic tests performed for other projects or tests reported in research, provided that in the opinion of the Manufacturer and Design Engineer there is sufficient basis for extrapolation to project conditions.

3. Extrapolation of previous test results beyond the limitations of Sections 8.6.3.7.4, 8.6.3.7.5.3 of FEMA 450 will not be permitted.

2.2 ACCEPTABLE MANUFACTURERS

A. The following manufacturers, which have successfully completed qualification testing of braces similar to those required for the project, will be considered acceptable manufacturers, subject to compliance with other requirements of the Construction Documents, including limitations on maximum brace dimensions.

1. ___________________________.

2. ___________________________.

3. ___________________________.

2.3 MATERIALS

A. Steel Core Plates: JIS G 3136 SN400 B, ASTM A36, or ASTM A572 Grade 42; except initial yield stress shall be 42 ksi, plus or minus 4 ksi, as evidenced by coupon testing of plates to be incorporated in work.

1. Plates 2 inches (50 mm) and thicker shall be supplied with Charpy V-Notch testing in accordance with ASTM A6 Supplementary Requirement S5, or approved equal. The impact test shall meet a minimum average value of 20 ft-lbs absorbed energy at +70 degrees F and shall be conducted in accordance with AISC Specification, or approved equal.

B. Casing: JIS G3466 STKR 400, or ASTM A500, Grade B. (Note that if qualified by testing, other casing materials can be used)

C. Welding Filler Material: Meet or exceed CVN toughness and elongation of material used for fabrication of tested assemblies.

1. H16 (maximum diffusible hydrogen), AWS A4.3.

D. Shop Primer: Manufacturer's standard zinc-rich rust preventative primer; containing less than 0.002% lead.

E. Fill Material: Manufacturer's standard cementitious grout; demonstrated suitable for function as a confining in-fill material by subassemblage qualification testing.

2.4 FABRICATION

A. Fabricate steel in accordance with Section _____.

1. Cut core plates to profile shown on Design Drawings. Conform to tolerances of Quality Assurance Manual, except tolerance on plate width shall not exceed plus or minus 0.2 inches (5 mm).

2. Splices in the steel core are not acceptable.

3. Roughness: After cutting, edges of core plates shall have roughness less than the surface roughness to which the tested BRBs were fabricated. Where no documentation is available to independently verify the surface roughness of the tested BRBs, edges of core plate shall have roughness less than or equal to sample 3 of AWS C4.1-77.

4. Gouges and Notches: Occasional gouges and notches less than 0.2 inches (5 mm) deep in edges of steel core may be repaired by grinding to a smooth transition. The length of transition shall be a minimum of 10 times the depth of gouge. The area shall be inspected by MT after grinding to ensure the entire depth of gouge has been removed. Deeper gouges shall be cause for rejection of steel core.

B. The maximum dimensions of the casing of the Buckling Restrained Brace shall be as indicated on the Drawings.

C. Connections: All holes for connections shall be manufactured using the same documented process employed in the manufacture of the tested BRBs. Where no documentation is available to independently verify the manufacturing process for holes in the tested BRBs, all holes for connections shall be drilled, and burrs removed in accordance with the AISC Code of Standard Practice.

D. Welding: Continuously weld joints, using procedures intended to minimize distortion.

E. Assembly: Assemble components of the Buckling Restrained Brace in a manner to ensure proper performance of the brace.

1. Examine core plates for straightness prior to contact with other material.

2. End confining plates shall be provided to ensure confinement of the in-fill material while allowing for movement of the steel core.

F. Finish: Prepare and paint unprotected metal surfaces of casing.

1. Solvent clean to remove oil and contaminants; Commercial Blast (SSPC-6) clean as minimum surface preparation.

2. Apply paint primer at a minimum dry film thickness of 3 mils (75 microns).

3. If faying surfaces of slip-critical bolted connections are painted, primer shall meet requirements of the RCSC (Research Council on Structural Connections) for a Class A coating.

2.5 SOURCE QUALITY CONTROL

A. Testing Laboratory will:

1. Review Manufacturer's Quality Assurance Plan, mill certificates and perform coupon testing.

2. Review Manufacturer's QA test and inspection reports.

3. Observe fabrication and assembly as requested by Engineer.

B. Contractor shall:

1. Notify Engineer no less than 30 days before the start of fabrication of the buckling restrained braces, to allow Engineer to observe fabrication and assembly process.

2. Perform testing and inspection in accordance with approved Quality Assurance Plan and requirements of QUALITY ASSURANCE – Section 1.5.

PART 3 - EXECUTION

3.1 ERECTION

A. Braces are erected under Section _____ – Structural Steel.

B. Prior to erection, clean faying surfaces of brace to be in contact with bolted connections to remove temporary coatings, applied for transport, and surface contaminants.

C. Buckling Restrained Brace members shall not be field cut or altered. Alterations to structural steel components to receive Buckling Restrained braces shall be subject to prior approval of Engineer.

D. No field welding to Buckling Restrained brace members will be permitted, including attachment of nonstructural components.

3.2 FIELD QUALITY CONTROL

A. Manufacturer's Representative will visit site to observe installation of Buckling Restrained Braces in accordance with Manufacturer's Quality Assurance Plan.

9. 2005 *SEISMIC PROVISIONS* AND OTHER TOPICS

9.1 2005 *Seismic Provisions*

All previous eight sections of this *Steel TIPS* report faithfully represent the published BRBF design requirements and as such stand as a snapshot of the BRBF body of knowledge as of July 2004. Also as of July 2004 rounds of balloting had been completed on the proposed 2005 edition of the *Seismic Provisions*. Between July 2004 and the publication of the 2005 *Seismic Provisions*, more rounds of balloting are planned. During the same period of time, analytical and experimental studies on BRBFs may be completed. As those studies are completed, proposals for revisions to the *Seismic Provisions* may be submitted to AISC for their consideration. This section is provided as a guide to the reader of requirements found in the *Seismic Provisions* but not in FEMA 450 and of proposals that may or may not materialize into future BRBF design requirements. All references to specifications sections are to the proposed 2005 edition of the *Seismic Provisions*.

Section 16.2b.5 defines adjusted brace strengths as a product of R_y. An exception to including R_y is defined where coupon tests or mill certificate information is used in defining P_{ysc}. Since the example of this *Steel TIPS* report defines both supplementary yield requirements on F_{ysc} and the performance of coupon tests, it was not necessary to incorporate R_y in the calculation of adjusted brace strengths, T_{MAX} and C_{MAX}. Furthermore, limiting the variability of F_{ysc} (but using the largest permissible F_{ysc} in calculating adjusted brace strengths) and requiring that coupon tests be performed appears to be a less conservative approach than to use Ry. For this example, by using the exemption, the largest $\beta\omega R_y F_{ysc} A_{sc}$ = $1.03x1.23x1.0x46xA_{sc}$ = $58.42A_{sc}$. Otherwise, we would have obtained $\beta\omega R_y F_{ysc} A_{sc}$ = $1.03x1.23x1.5x36xA_{sc} = 68.41A_{sc}$.

Section 16.3a increased by 10% the required strength of bracing connections. The proposed *Seismic Provisions* includes a 1.1 factor while FEMA 450 does not. The 1.1 factor is reasonable for connection design.

Section 16.4a.2 deletes the beam stiffness check required by FEMA 450 section 8.6.3.4.1.2.

Section 16.7 represents a new section that defines the protected zone.

Throughout the entirety of section 16 a 2.0 factor replaces the 1.5 factor found in FEMA 450 wherever "times the Design Story Drift" is referenced. There is a possibility that a proposal to increase the 2.0 factor to some higher value may be presented to the *Seismic Provisions* committee. It is currently being debated whether for an elastic, force-based analysis (such as the one performed in this *Steel TIPS* report) $2.0\Delta_{bm}$ represents enough of an amplifier to estimate local demands given elastic story drift results based on a global parameter R. The concern over a $2.0\Delta_{bm}$ factor is that it may underestimate brace demands. To try to quantify the effect of a $2.0\Delta_{bm}$ factor on deformation and force demands Tables 16 through 18 are presented.

Table 16. Frame BF-1 BRB Ductility Demands

Study	BRB design		Δ_{bx}/Δ_{by}	Ductility Demand
	ρ	ϕ		
Sabelli (2001)	1.0	1.0	1.0	13.6 for 3vb2 14.5 for 6vb2 12.9 for 6vb3 (μ + σ)
Fahnestock et al., (2003)	1.0	1.0	1.0	13.1 (μ + σ)
This *Steel TIPS* report	1.11	0.9	0.64 (max)	6.39 (max)

Table 17. Frame BF-2 BRB Ductility Demands

Study	BRB design		Δ_{bx}/Δ_{by}	Ductility Demand
	ρ	ϕ		
Sabelli (2001)	1.0	1.0	1.0	13.6 for 3vb2 14.5 for 6vb2 12.9 for 6vb3 (μ + σ)
Fahnestock et al., (2003)	1.0	1.0	1.0	13.1 (μ + σ)
This *Steel TIPS* report	1.23	0.9	0.52 (max)	5.23 (max)

Table 18. Strength Adjustment Factors

Braced Frame	Fahnestock (2004)	This report	Fahnestock (2004)	This report
	ω (μ + σ)	ω max	ωβ (μ + σ)	ωβ max
BF-1 (Table 8)	1.14	1.23	1.27	1.27
BF-2 (Table 11)	1.14	1.18	1.27	1.20

From studying Tables 16 through 18 it becomes clear that an elastic, force-based analysis that utilizes $2.0\Delta_{bm}$ as a factor to estimate local demands results in as good an estimate of force demands but not as good an estimate of ductility demands as would be obtained from a nonlinear dynamic analysis. The arguments in favor of or against changing the $2.0\Delta_{bm}$ factor and the role of rho, phi, etc. are beyond the scope of this *Steel TIPS* report and will not be discussed. However, it is a fact that analytical studies conducted to date are based on sizing the braces with no overstrength and that is not the case in day-to-day designs such as the one in this *Steel TIPS* report. Such a difference in design approaches needs to be considered when discussing the $2.0\Delta_{bm}$ factor.

Currently there is no separate displacement protocol for subassemblage testing. The idea of having a subassemblage protocol separate from the uniaxial protocol has been considered before and may materialize in the following year. Analytical studies (Sabelli, 2001; Fahnestock et al., 2003) do not characterize BRBF demands in terms of total brace end rotations and therefore there is nothing to compare against. Previous experimental studies (López et al., 2002; Merrit et al., 2003a, 2003b; SIE, 2003) did not use brace end rotations as the controlling test variable. A subassemblage test of a large-capacity brace currently being planned uses interstory drift ratio as the controlling variable. Furthermore, total brace end rotations are not quantities normally extracted during the course of an elastic analysis. When the *Recommended Provisions* were originally written there were valid reasons to be skeptical about the behavior of BRBs subjected to concurrent flexural and axial strains, and thus total brace and rotation seemed to be an appropriate variable to characterize. Since the *Recommended Provisions* were last officially updated by the joint AISC/SEAOC committee, late 2001, numerous subassemblage tests have been performed (López et al., 2002; Merrit et al., 2003a, 2003b; SIE, 2003). Now that a larger body of knowledge is available, and BRBs do not appear to be as sensitive to concurrent axial and flexural strains as once thought, departing from total brace end rotation as a controlling parameter may be worth considering.

9.2 Other topics

This section contains suggestions for consideration by the structural engineer.

Currently the only way of justifying using $1.0\Delta_{bm}$ in calculating brace deformations, strains, and adjusted brace strengths is by performing nonlinear dynamic analyses. If approved by the building official, subject to qualified peer review, performance of nonlinear static analyses may be used as a justification for using $1.0\Delta_{bm}$ in calculating brace deformations, strains, and adjusted brace strengths.

Include in the drawings a table defining required BRB deformation capacities. The structural engineer should decide whether to define deformation capacities larger than the currently required $2.0\Delta_{bm}$. It is expected that requiring deformation capacities approaching the results from analytical studies should not increase the BRB cost. However, the structural engineer is strongly advised to consult with BRB manufacturers before making such a decision.

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About the authors….

Walterio A. López, S.E., is an Associate at Rutherford & Chekene. He is very involved in the codification, building design, and laboratory testing of buckling-restrained braced frames (BRBFs). He has authored and coauthored several technical articles concerning this system. He served on the joint AISC/SEAOC committee that developed the first design provisions for buckling-restrained braced frames. He is now the chair of the Steel Seismology Committee of the Structural Engineers Association of California.

He can be reached at:

Walterio A. López, S.E
Rutherford & Chekene
427 Thirteenth Street
Oakland, CA 94612
Phone: (510) 740-3200, Fax: (510) 740-3340
E-mail: wlopez@ruthchek.com
Web page: www.ruthchek.com

Rafael Sabelli, S.E., is Director of Technical Development at DASSE Design. He is a member of the AISC Committee on Seismic Provisions for Structural Steel Buildings. He served as chair of the Seismology and Structural Standards Committee of the Structural Engineers Association of Northern California (SEAONC); he is currently the secretary of SEAONC. He was the 2000 EERI/FEMA NEHRP Professional Fellow. He chaired the joint AISC/SEAOC committee that developed the first design provisions for buckling-restrained braced frames.

He can be reached at:

Rafael Sabelli., S.E.
33 New Montgomery Street, Suite 850
San Francisco, CA 94105
Phone: (415) 243-8400, Fax: (415) 243-9165
E-mail: Sabelli@dasse.com
Web page: www.dasse.com

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Funding for this publication provided by the California Field Iron Workers Administrative Trust.