ELSEVIER
Available online at www.sciencedirect.com
ScienceDirect
Procedia Engineering 14 (2011) 2187–2195
Procedia Engineering
www.elsevier.com/locate/procedia

The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction

Performance Assessment of Buckling Restrained Braces

H. Y. CHANG[1a] and C. K. Chiu[2]

[1]*Department of Civil and Environmental Engineering, National University of Kaohsiung, Taiwan*
[2]*Department of Construction Engineering, National Taiwan University of Science and Technology, Taiwan*

Abstract

In the company paper, the global drift capacity and demands of a 6-story steel office building adopted with buckling restrained braces (BRBs) have been evaluated. This paper evaluates the seismic performance of the building from a local perspective. In detail, the capacity and demands of the BRBs were assessed by test data and response analysis. Then, the level of confidence was computed against the potential of BRB yielding, buckling and fracture failures. The result shows that the BRBs can provide a high level of confidence, ensuring the building to achieve the performance objectives of immediate occupancy and life safety. But in meeting the performance objective of collapse prevention, the confidence level may be far smaller than the 50%-value recommended by FEMA 351. It suggests a necessity of more carefully assessing the seismic vulnerability of braced steel frames, especially for collapse prevention and from a local perspective.
Keywords: Confidence level, performance evaluation, local failures, buckling restrained brace.

Keywords: Confidence level, performance evaluation, local failures, buckling restrained brace.

[a] Corresponding author: Email: hychang@nuk.edu.tw

doi:10.1016/j.proeng.2011.07.275

1. INTRODUCTION

Toward the development of performance-based engineering, there has been a necessity of assessing structural performance on a reliability basis. FEMA 351, for example, recommends design solutions to provide a 50 % level of confidence against the potential of local failures. In the company paper, the global drift demand and capacity of a 6-story steel office buildings adopted with buckling restrained braces (BRBs) have been evaluated (Chang 2009). This paper assesses the level of confidence against BRB yielding, buckling and fracture failures. All that helps gain a better understanding about the global and local performance of steel buildings adopted with BRBs.

2. STUDIED BRB MEMBERS

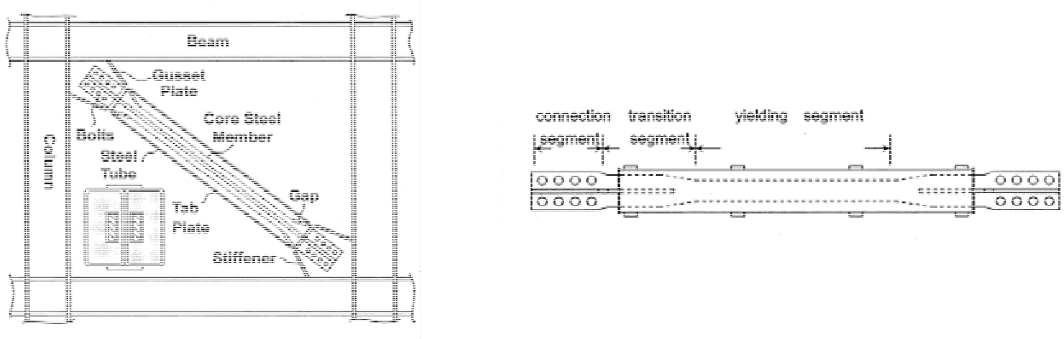
2.1. Performance objectives

Table 1 gives an example illustrating the structural performance levels and damage of braced steel frames. As can be seen, BRB deformations at first yield, buckle and fracture are important to justify the performance levels of braced steel frames.

Table 1: Structural performance levels and damage (FEMA 356)

Elements	Type	Structural performance levels		
		Collapse Prevention (C.P.)	Life Safety (L.S.)	Immediate Occupancy (I.O.)
Braced steel frames	Primary	Extensive yielding and buckling of braces. Many braces and their connections may fail.	Extensive braces yield or buckle but may not totally fail. Many connections may fail.	Minor yielding or buckling of braces.
	Secondary	Same as primary	Same as primary	Same as primary
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent

2.2. BRB mechanism



(a) Double-cored BRB member (b) BRB core plate

Figure 1: A steel frame adopted with a double-cored BRB and the core plate of the BRB

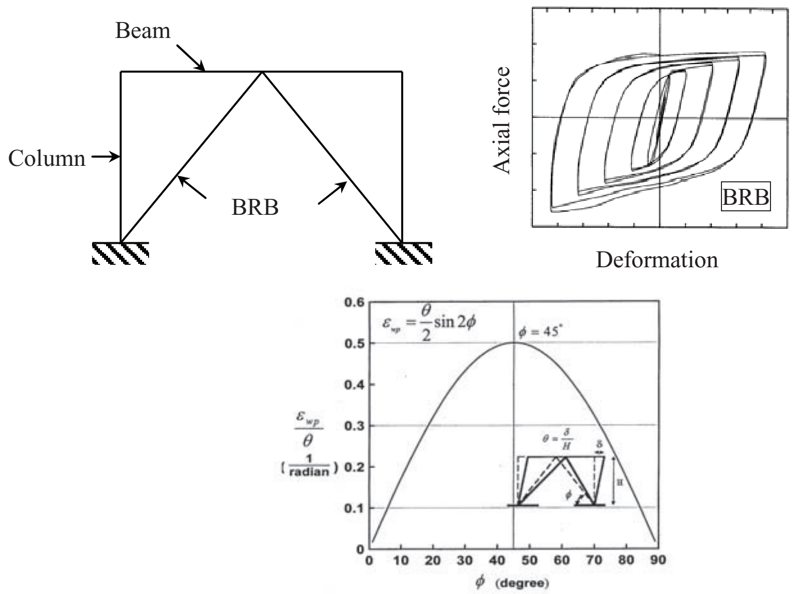


Figure 2: Configuration of a chevron BRBF, hysteresis loops and BRB strain and story drift

To reduce the length and number of bolts in the brace-to-gusset connection, the double-cored buckling restrained braces (DCBRBs) have been developed and extensively tested in Taiwan in the past few years. Figure 1 gives the details about the application of a double-cored BRB to a steel frame and the details of the BRB core plate.

A_c is the area of core plate and F_y is steel nominal yield stress. Ω and Ω_h respectively denote over-strength factor and strain hardening factor (Ω =1.5 for A36 steel, Ω =1.1 for A572 Gr. 50 steel; Ω_h =1.5 for A36 steel, Ω_h =1.3 for A572 Gr. 50 steel). β is an adjustment factor for the difference between tensile and compressive strength (~1.1).The yield strength and ultimate strength of a BRB can be estimated as follows

$$P_y = \Omega A_c F_y \tag{1}$$

$$P_{\max} = \Omega_h \beta P_y \tag{2}$$

E is Young's modulus. A_t and A_j respectively denote the areas of transition segment and connection segment. L_c , L_t and L_j respectively represent the length of core plate (yield segment), transition segment and connection segment. The stiffness of a BRB can be evaluated using the following equation.

$$\frac{1}{K_{eff}} = \frac{1}{K_c} + \frac{1}{K_t} + \frac{1}{K_j} = \frac{L_c A_t A_j + L_t A_c A_j + L_j A_c A_t}{E A_c A_t A_j} \tag{3}$$

α is energy concentration factor (α =0.5) and L_{wp} is the distance between work points (see Figure 1(b)). The core strain ε_c can be evaluated using the following equations.

$$\varepsilon_c = \varepsilon_{wp} / \alpha \tag{4}$$

$$\alpha = L_c / L_{wp} \tag{5}$$

Figure 2 shows the configuration of a chevron BRBF, hysteresis loops and BRB strain and story drift. For a chevron BRBF, if the beam deformation is ignored, the BRB average strain ε_{wp} can be estimated by using the equation of story drift angle θ and brace slope ϕ

$$\varepsilon_{wp} = \frac{\theta}{2}\sin(2\phi) \tag{6}$$

2.3. Deformation criteria

From equations (1) and (3), BRB deformations at first yield can be estimated as

$$D_y = P_y / K_{eff} \tag{7}$$

BRB members tend to develop the maximum strength when buckling failures occur. From experimental statistics, the ratio of post-yield stiffness to elastic stiffness was found to have an average of 0.05. BRB deformations at buckling can therefore be calculated as

$$D_m = D_y + (P_{\max} - P_y)/(0.05K_{eff}) \tag{8}$$

A regression analysis has been made on the result of fracture tests on 26 BRB members. The maximum deformation D_f ,which a BRB member can develop before fracture, can predicted by the following equations,

$$D_f = 2.78(D_{ave} / D_y)^{0.65} \cdot D_y \tag{9}$$

$$D_{ave} = CPD / 4N_f \tag{10}$$

In the above equation, Rain Flow Theory has been used to count the number of plastic loading cycles N_f. In addition, the average plastic deformation D_{ave} is defined as the ratio of accumulative plastic deformation CPD to $4N_f$.

2.4. BRB properties

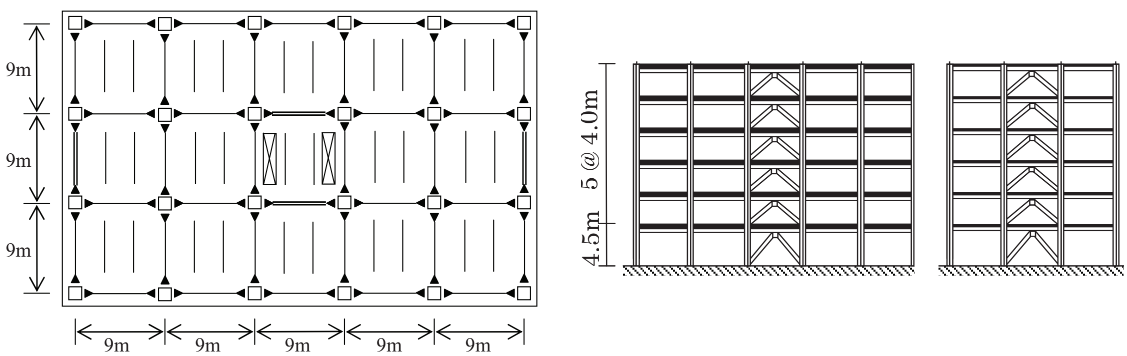
The analyzed BRB members were originally designed to adopt in a 6-story steel office building (Chang 2009). Figure 3 depicts the floor plan and elevation of the building. The building uses A572 Grade 50 steel built-up box columns and braces, and A36 steel H-shaped beams. Table 2 summarizes the member sizes.

The earthquake response of the building has been simulated via a platform of inelastic structural analysis for 3D systems (Lin and Tsai 2006). The building has a fundamental period of 0.99 sec and it translates in the Z-direction at the first vibration mode. In the analysis, the frame was excited in the Z-direction. The beams and columns were simulated utilizing plastic hinge models with bi-linear stress-strain relation. The interaction surface of axial force and bending moment were also considered in the columns.

The BRBs were replicated utilizing truss elements with the two-surface plastic hardening rule. The BRB average strain analyzed was used to estimate the BRB core strain with equations (4) and (5). The largest deformation demands were found on the BRBs of the 2nd-story structure. In the following, the seismic performance of the BRBs was selected to study in detail.

Table 2: Member size of example building

Story	Column	BRB_X	BRB_Z	Story	Girder_X	Girder_Z	
6F	□-450×19	Ac=25 cm^2	Ac=35 cm^2	RF	H488×300×11×18	H386×299×9×14	H386×299×9×14
5F	□-450×19	Ac=25 cm^2	Ac=35 cm^2	6F	H488×300×11×18	H386×299×9×14	H386×299×9×14
4F	□-450×22	Ac=50 cm^2	Ac=55 cm^2	5F	H488×300×11×18	H390×300×10×16	H400×300×11×18
3F	□-450×22	Ac=50 cm^2	Ac=55 cm^2	4F	H488×300×11×18	H390×300×10×16	H400×300×11×18
2F	□-450×25	Ac=50 cm^2	Ac=55 cm^2	3F	H582×300×12×17	H390×300×10×16	H400×300×11×18
1F	□-450×25	Ac=60 cm^2	Ac=70 cm^2	2F	H582×300×12×17	H390×300×10×16	H594×302×14×23



(a) Floor plan (bold line: braced frame)

(b) Elevation

Figure 3: Floor plan and elevation of example building

3. CONFIDENCE ASSESSMENT

A confidence parameter, λ, has been used to associated with the probability that a structure will satisfy a definite performance objective for a specific hazard (Yun et al 2002),

$$\lambda = \frac{\gamma \gamma_a D}{\phi_U \phi_R C} \tag{11}$$

In equation (1), C and D are median estimates of structural capacity and demand. γ is demand uncertainty factor that principally accounts for uncertainty inherent in prediction of demand arising from variability in ground motion and structural response to that ground motion; γ_a is analysis uncertainty factor that accounts for bias and uncertainty associated with specific analytical procedures used to estimate structural demand; ϕ_U and ϕ_R are resistance factors that accounts for the uncertainty and randomness inherent in prediction of structural capacity.

The C.L. can be determined using the λ value by a back calculation to obtain the standard Gaussian variate K_x associated with probability of x not being exceeded found in conventional probability tables.

$$\lambda = e^{-\beta_{UT}(K_x - k\beta_{UT}/2b)} \tag{12}$$

$$\beta_{UT} = \sqrt{\sum_i \beta_i^2} \tag{13}$$

In equations (12) and (13), β_{UT} is associated with the uncertainty about estimating structural demand and capacity (see the details in 3.2 and 3.3). k is the logarithmic slope of the hazard curve at the desired hazard level (refer to 3.1). b represents the change in demand as a function of ground motion intensity (refer to 2.4).

As mentioned, the assessment of C.L. (or the calculation of λ) requires estimating site-specific seismic hazard, structural demand and capacity. The following sections steps through the assumptions used to calculate the coefficients in the two equations for the studied steel frame and brace members.

3.1. Site-specific Seismic Hazard

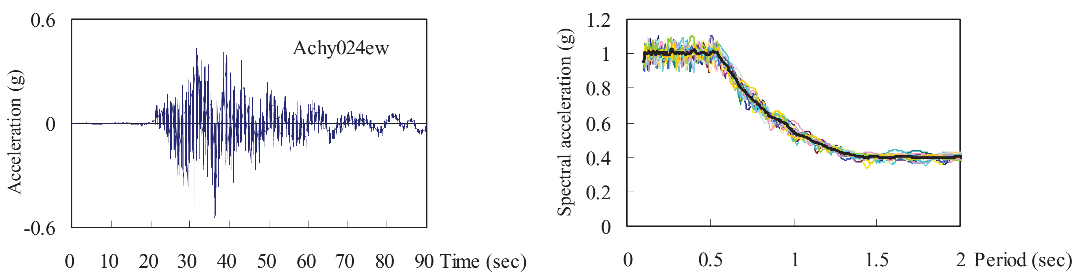


Figure 4: Acceleration time history and elastic response spectra (5% damping ratio)

A set of 20 artificially generated ground motions was adopted in the analysis. Figure 4 gives an example illustrating the acceleration time history and elastic response spectra. The phase contents of the ground motions were simulated by using the data actually recorded around the building site. Throughout iterative procedures, the amplitude spectra of the ground motions were proportionally fitted to that of the maximum considered earthquake (MCE), corresponding to the 2% in 50-year probability of exceedance. After that, the ground motions were scaled by multiplying scalar factors of 0.80 and 0.33, respectively corresponding to the 10% and 50% in 50-year probability of exceedance.

The logarithmic slope of the hazard curve at the desired hazard level, k , is used in the evaluation of the resistance factors ϕ , demand factors γ and confidence levels (C.L.). The hazard curve is a plot of the probability of exceedance of a spectral amplitude value, H_{Si} , versus the spectral amplitude for a given response period, S_i .

$$H_{Si}(Si) = k_0 S_i^{-k} \tag{14}$$

In equation (3), k_0 is a constant. Given the spectral acceleration values at 10%/50 year and 2%/50 year exceedance probabilities, for example, the value of k can be calculated as

$$k = \frac{\ln\left(\frac{H_{s1(10/50)}}{H_{s1(2/50)}}\right)}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} = \frac{1.65}{\ln\left(\frac{S_{1(2/50)}}{S_{1(10/50)}}\right)} \approx 7.40 \tag{15}$$

3.2. Structural Capacity and Resistance Factors

The median estimates of deformation capacities and resistance factors for BRB members tested using the AISC loading protocol are summarized in Table 3. Structural capacity and resistance factors needs evaluating for a specific limit state. For the studied BRB members, first yield, buckling and fracture are the three limit states to consider. The deformation capacity C and uncertainty parameter β_{UC} have been evaluated by experimental statistics and theoretical prediction (Chang and Huang 2010). It was found that the deformation capacities of BRB members, C, could be theoretically predicted. For example, The BRB capacities against fracture failures were evaluated using equations (9) and (10). In the equations, the number of load cycle and BRB average strain were estimated by nonlinear time history analysis (see the details in 2.4 and 3.3).

β_{UC} is the standard deviation of the natural log of BRB deformation capacity due to uncertainty. For first yield and fracture failure, as illustrated by Table 3, β_{UC} is 0.10 and 0.15, respectively. The ratios of post-yield stiffness to elastic stiffness were found to have a large variation, and significantly increased the uncertainty about predicting the deformation at buckling. For simplicity, β_{UC} is assumed to be 0.4 for buckling. Following that, the resistance uncertainty factor ϕ_U is calculated as follows,

$$\phi_U = e^{-k\beta_{UC}^2/2b} \tag{16}$$

β_{RC} is the standard deviation of the natural log of BRB deformation capacity due to randomness. β_{RC} can be assumed to be 0.20. Following that, the resistance variability factor ϕ_R is calculated as

$$\phi_R = e^{-k\beta_{RC}^2/2b} \tag{17}$$

Table 3: BRB deformation capacities and resistance factors (b =0.813, k =7.40)

Failure mode	C (%)	β_{UC}	ϕ_U	β_{RC}	ϕ_R
Yield	0.19	0.10	0.96	0.20	0.83
Buckling	1.82	0.40*	0.48	0.20	0.83
Fracture	0.77	0.15	0.90	0.18	0.86

Table 4: BRB deformation demands and demand factors (b =0.813, k =7.40)

Seismic hazard	D (%)	β_a	γ_a	β_{RD}	γ
50%/50 year	0.46	0.17	1.14	0.20	1.20
10%/50 year	0.59	0.23	1.27	0.20	1.20
2%/50 year	0.80	0.24	1.30	0.20	1.20

Table 5: Calculation of confidence levels for BRB members (b =0.813, k =7.40)

Performance level	Failure mode	λ	β_{UT}	K_x	C.L.
I.O.	Yield	4.16	0.28	-3.82	<1%
	Buckling	0.87	0.48	2.47	99.3%
L.S.	Fracture	1.51	0.34	0.34	63.1%
	Buckling	1.24	0.50	1.85	96.8%
C.P.	Fracture	2.10	0.35	-0.53	29.9%
	Buckling	1.72	0.51	1.26	89.6%

3.3. Structural Demand and Demand Factors

The median estimates of deformation demands and demand factors for BRB members are summarized in Table 4. Structural demands and demand factors needs evaluating at a definite hazard level. For the studied steel frame and BRB members, the 2%, 10% and 50% in 50-year probability of exceedance are the three levels of seismic hazard to consider. The deformation demand D and variability parameter β_{RD} have been evaluated using nonlinear time history analysis (Chang 2009). Since the example building is not located near a known fault, the variability in excitation orientation is not considered. The β_{RD} value is calculated by taking the standard deviation of the log of the maximum story drifts evaluated for each of the 20 accelerograms referred above. Therefore β_{RD} only reflects the variability in acceleration. Following that, the demand factor γ is calculated as

$$\gamma = e^{k\beta_{RD}^2/2b} \tag{18}$$

The parameter β_a considers the uncertainties about analysis procedures, bias factor, damping, live load and material properties. The uncertainties about damping, live load and material properties are negligible, when compared to those about the analysis procedure and bias factor. The uncertainty about analysis procedures is the extent that the benchmark, nonlinear time history analysis represents actual physical behavior. The bias factor for each procedure is calculated as the ratio of the median demand resulting from nonlinear time history analysis divided by that from the other analysis procedure. Based on the judgment and understanding of the relative importance of strength degrading, P-delta effects, and phenomena not well considered for in the analysis, β_a is assumed to be 0.20 for the studied steel frame and BRB members. Following that, the demand uncertainty factor γ_a is calculated as

$$\gamma_a = e^{k\beta_a^2/2b} \tag{19}$$

4. SUMMARY AND CONCLUSION

The calculation of confidence levels are summarized in Table 5. For IO, as illustrated by Table 1, the BRBs are allowed to yielding or buckling. The level of confidence therefore has been calculated against the potential of BRB yielding and buckling failures. Accordingly, for LS and CP, the level of confidence has been calculated against the potential of buckling and fracture failures.

From Tables 3 and 4, it is clear that the BRBs have enough capacity against buckling failures when the building approaches the IO limit state. It can also be found that the BRBs have large capacity against fracture failures when the building approaches the LS limit state. Therefore, as can be seen in Table 5, the BRBs can provide a high level of confidence that ensure the building to satisfy the desired IO and LS performance.

When the building approaches the CP limit state, however, the level of confidence against the potential of BRB fracture failures may become far smaller than the 50%-value recommended by FEMA 351 (see Table 5). It means the BRBs cannot provide an appropriate level of confidence in meeting the performance objective of CP. The result has suggested a necessity of more carefully assessing the seismic vulnerability of braced steel frames, especially for collapse prevention and from a local perspective.

ACKNOWLEDGMENTS

The author greatly thank National Center for Research on Earthquake Engineering (NCREE) in Taiwan for providing the BRB test data and artificially ground motions analyzed in this paper.

REFERENCES

[1] FEMA 356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, D.C.

[2] Chang HY. (2009). Seismic fragility assessment of buckling restrained braced frames. Proceedings of the 11th Taiwan-Korea-Japan Joint Seminar on Earthquake Engineering for Building Structures (SEEBUS), Kyoto, Japan.

[3] Chang HY and Huang YJ. (2010). An investigation of BRB experimental performance. Proceedings of the National Conference on Structural Engineering, Taoyuan, Taiwan (in Chinese).

[4] Yun SY, Hamburger RO, Cornell CA and Foutch DA. (2002). Seismic performance evaluation for steel moment frames. Journal of Structural Engineering, Vol. 128, No.4, pp. 534-545.

[5] Lin BZ and Tsai KC. (2006). Platform of inelastic structural analysis for 3D systems - PISA3D R2.0.2 users' manual. Taipei, National Center for Research on Earthquake Engineering (NCREE).