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Seismic Performance of Buckling Restrained Bracing System in Steel Structure

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Abstract: *This paper describes the seismic performance of buckling restrained braces (BRB) which intend to buckle and yield under seismic action in Steel structures. In this paper, lateral load resisting system taken as concentric bracing system of V and inverted V type bracing will be provided in the structure. BRB has components of in-filled steel core of cement mortar and steel casing. During seismic, the bracing member will subject to compression and tension, also intend to carry the moment from the frame for which they were not designed. To mitigate these effects and increase the efficiency of the BRB in performance, the comparisons will be made for the response between the type of bracing provided through which a novel connection method was suggested. The modelling and analysis made by help of software and also design calculations will be done.*
Keywords: *buckling restrained braces, concentric bracing, steel structures, seismic performance*

I. Introduction

Buckling restrained braced frames (BRBF) have been used as lateral load resisting member. Under large earthquake motions, the stable hysteretic behaviour of the buckling restrained braces (BRBs), which yield in both tension and compression, provides substantial energy dissipation and ductility. This behaviour has been tested and well documented in several experimental studies and there are clauses for the design of BRBFs in the *2016 AISC Seismic Provisions for Steel Buildings.*

The modelling and analysis will be performed with the complete BRBFs with gusset plate connected to the beams and columns, rather than analysis involving only BRBs and gusset. Gusset performance was important to the performance of BRBF system. Adequate gusset design must be made for the various bracing cases in accordance to the *2016 AISC Seismic Provisions for Steel Buildings.*

This BRBs constitute of steel tube in-filled by cement mortar and inner core of the member consists of steel plate which was connected to the gusset plate to resist buckling and yielding of member. And also BRBFs will have V type bracing and inverted V type bracing for which the static load cases will be applied and the behaviour of BRBs will be resulted. This part describes the detailed concepts and evolutions of BRBFs from the limitation of usage of concentric bracing system to the structure.

1.1 Limitations to classical concentric braces (CBs):

The most commonly used technique for seismic protection of structures is concentric steel braces. However, ordinary concentric braces are characterised by a limited ductility capacity under cyclic loading. And also, their response is not symmetric and exhibits substantial strength degradation, which causes buckling of braces under strong compression. Another complexity originates as, deviation from the predicted response of overall structure equipped with concentric braces based on conventional models to the actual distributions of internal forces and deformations in the overall structure (Jain and Goel, 1979; Khatib and Mahin, 1987).

For example, in common double T sections, required strength and stiffness are difficult to be tailored, thus giving rise very often to some “weak or soft stories”. Finally, out-of-plane buckling of braces could cause severe damage to nonstructural components.

These problems in the seismic design of concentric braces have long been investigated. Research has been carried out in the United States, Canada and Japan, in order to limit the flexural slenderness of the brace to a small value, so as to be compatible with the ductility requirements associated with the adopted strength reduction factor. And also addressed to solve the problem of damage concentration in a few weak stories, by means of alternative structural schemes, such as “Zip” frame configuration proposed by Khatib and Mahin (1987). The substitution of bare steel braces with composite ones has also been suggested for improving the available ductility (Liu and Goel, 1987).

When ordinary braces are used for strengthening existing constructions, such as when they are added to existing RC frames, the problem of the introduction of additional axial forces in the columns of the existing frames also needs to be considered. This problem is intense when the brace slenderness is kept low in order to limit the strength degradation of the brace under compression.

1.2 Buckling Restrained Braced System:

1.2.1 Concept:

BRBs have been considered to solve the above problems. BRBs are characterised by the absence of buckling which allows a stable hysteric behaviour, different from steel concentric braces. And also, BRBs permit an independent design of stiffness, strength and ductility properties. This kind of behaviour is achieved by limiting the buckling of the steel core within the bracing elements. The axial strength is separated from the flexural buckling resistance. In fact, the axial load is confined to the steel core, while the buckling restraining mechanism does not permit the overall brace buckling and restrains the high-mode steel core buckling.

1.2.2 History:

The concepts of BRBs were first introduced in 1980s at the Tokyo Institute of Technology in Japan. The first BRB of moment resisting frame was built in Japan in 1988 with the use of "unbonded brace" manufactured by Nippon Steel Corporation and consecutively starting from 1988, nearly 300 buildings have been equipped. After several tests carried out, in 1999 at the University of California, Berkeley, the technology has also been implemented in the United States by utilising BRBs for the seismic retrofitting of the UC Davis Plant and Environmental Sciences. BRB framed systems have been approved for the use in United States by The Division of the State Architects (DSA), The Office of Statewide Health Planning and Development (OSHPD), The University of California. To date there are an estimated 150 structures in the United States have been equipped with 20,000 BRBs (Lopez, 2008).

1.2.3 Manufacturer:

Currently there are three proprietary manufactures of BRBs are "Nippon steel corporation, Star seismic and Core brace".

1.2.4 Research:

The BRB technology is currently undergoing strong development, with a growing number of buildings using buckling restrained braces as a primary lateral force resisting system. This strong development is also testified by several research studies that are ongoing in the United States, Taiwan and Japan (Tsai et al., 2004a; Sabelli and Aiken, 2004; Wada and Nakashima, 2004).

Different types of BRBs have been studied, all based on the same concept of using tubes or other elements for restraining lateral displacements while allowing axial deformations of the core. In the most classical form, the restraining element is a steel tube filled with concrete. A layer of unbonding material is placed at the contact surface between the core plates and the filling concrete, this version therefore being called the "unbonded brace". The unbonding material allows the brace to slide freely inside the buckling restraining unit and lets the transverse expansion of the brace to take place when the core yields in compression.

1.2.5 Evolution:

"Only-steel" solutions have also been proposed, with two or more steel tubes in direct contact with the yielding steel plates. In the case of steel tubes connected very nearer to each other (latter), the restraining tubes can also be connected by bolted steel connections, thus allowing an easy inspection and maintenance during the lifetime or after a damaging earthquake (Tsai et al., 2004a). An adequate gap between the brace and the restraining tubes is also required in case of "only-steel" BRBs, in order to provide the necessary space for relative deformation between both member components.

Yielding of this special type of bracing occurs when the plastic strength of the core steel plates is achieved. The axial stiffness is determined by the combination of two springs in series having the axial stiffness of the internal core and terminal tapered plates. The length and size of the latter can be independently fixed to some extent. In any case, the possibility of avoiding compression buckling allows very slender steel plates to be used as the core of the BRB, with a relatively low plastic strength and without impairing the system ductility. In this way, the yielding of the BRB can be regulated to very low inter story drifts, thus permitting the dissipative action to be activated soon.

Low cycle fatigue (failure) characteristics have been shown to depend on a variety of factors, including the restraining mechanism used, material properties, local detailing, workmanship, loading conditions and so on. Inelastic deformation (ductility) capacities are generally quite large, with cumulative cyclic inelastic deformations often exceeding 300 times the initial yield deformation of the brace before failure (Nakamura et al., 2000).

1.2.6 Principle:

Anyway, as mentioned above, the basic principle, which characterises the BRB response, is based on the possibility of decoupling of axial-resisting and flexural-resisting aspects in the compression field. In fact a steel core plate has to resist axial stresses, while buckling resistance is provided by an external sleeve, which may be made of steel, concrete, or composite.

1.2.7 Components:

BRB-bracing member has steel core which is possible to divide the core into three zones:

- The yielding zone, with a reduced cross-section area, inside of the zone of lateral restraining provided by the sleeve (zone C)
- The transition zones, with a larger area than the yielding zone, but similarly restrained (zone B)
- The connection zones, which extend outside the sleeve and connect to the frame by means of gusset plates (zone A)

1.2.8 End-connections:

In order to prevent the instability of the lateral restraint (sleeve) and to permit the full axial yielding of the steel core, the end-connections of these devices have to be able to transfer forces to the core without the development of a significant stress state in the sleeve. Furthermore, the end-connections have to be designed to avoid modes of overall instability of the bracing member.
In addition, special end details must be designed in order to permit inelastic deformations of the steel core.

II. Modeling

2.1 SEVEN-STORY OFFICE BUILDING:

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.

2.2 Building Model 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 report, the use of the same building model is intended to provide a point of reference for comparison of different braced-frame systems. Figures 2.1, 2.2 and 2.3 defines the Inverted V braces and Figures 2.1, 2.4 and 2.5 defines the Chevron V braces.

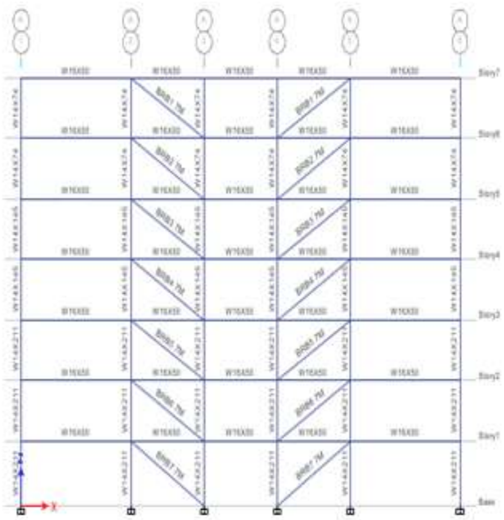

Figure 2.1. Framing PLAN

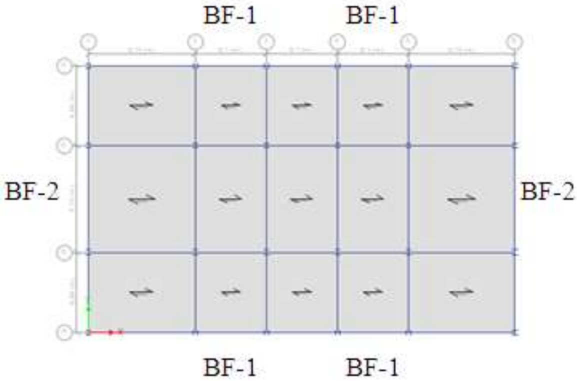

Figure 2.2. Typical BF-1 elevation

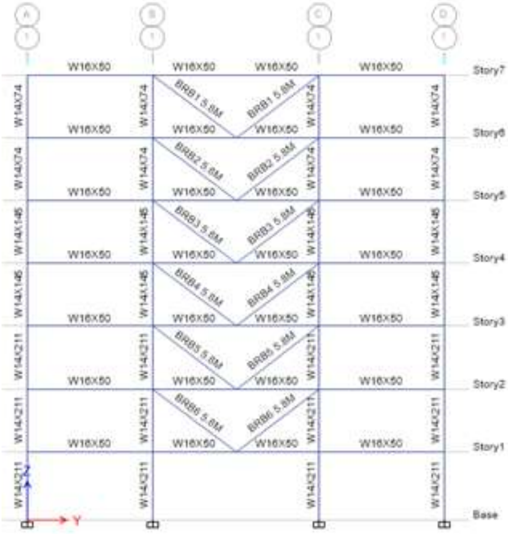

Figure 2.3. typical BF-2 elevation

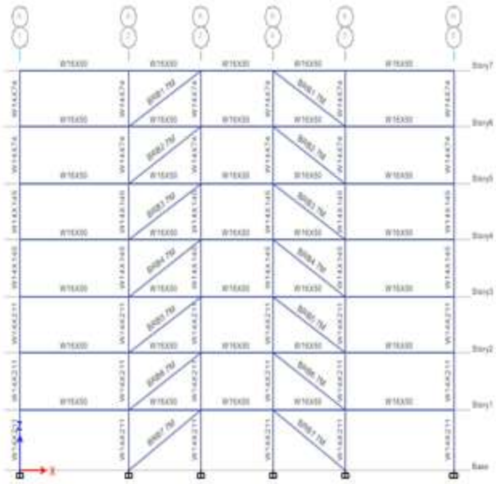

Figure 2.4. Typical BF-1 elevation

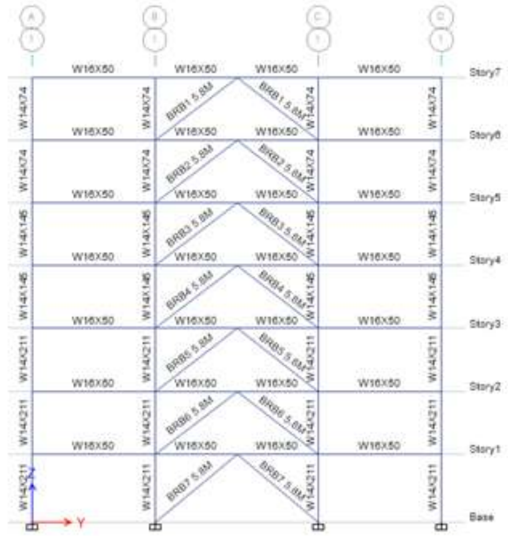

Figure 2.5. typical BF-2 elevation

2.3 Structural Materials:

- W sections - ASTM A992

(F_y = 345 N/mm^2, F_u = 448.16 N/mm^2)

- BRB Steel Core - ASTM A36 with supplemental yield requirements: F_{ysc} = 290 N/mm^2 (±27.5 N/mm^2). Coupon tests required.
- BRB Steel Casing - ASTM A500 Grade B
- Gusset plates - ASTM A572, Grade 50

(F_{yg} = 345 N/mm^2, F_u = 448.16 N/mm^2)

- Lightweight concrete fill - f_c' = 20.68 N/mm^2

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 A325Design 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

2.4 Loading:

Roof Loading:

Roofing and insulation	0.34 kN/m^2
Steel deck + Fill	2.25
Steel framing and fireproofing	0.4
Ceiling	0.15
Mechanical/Electrical	0.1
Total	3.21 kN/m^2

Floor Weights:

Steel deck + Fill	2.25 kN/m^2
Steel framing and fireproofing	0.60
Partition walls	0.95
Ceiling	0.15
Mechanical/Electrical	0.1
Total	4.0 kN/m^2

Average Exterior Curtain Wall Weight including Column and Spandrel Covers: 0.72 kN/m^2

Live Loads:

Roof	0.95 kN/m^2
Floor	2.40 kN/m^2

2.5 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, Fa = 1.0, Fv = 1.5, and the Maximum Credible Earthquake (MCE) parameters given in Table 1 are obtained

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

Table 1. Site Parameters

The response spectrum is constructed per section 9.4.1.2.6 of ASCE 7-02 and shown in Figure 2.4. Throughout this report all equations and section references are for ASCE 7-02 unless otherwise noted.

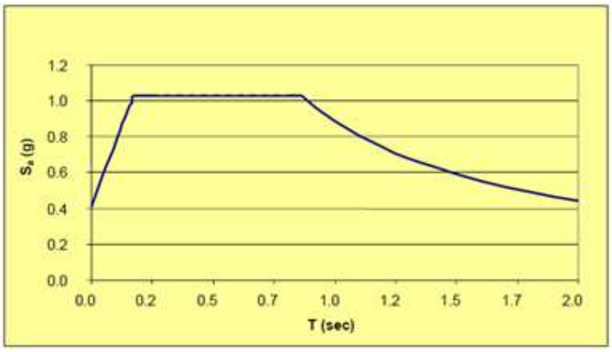

Figure 2.4. Design Response Spectrum

2.6 Seismic Load Resisting System Parameters:

The values of R, Cd, Ct and x listed in Table 2 are found in Chapter 4 of FEMA 450.

Table 2. System Parameters

Parameter	Value
Building Height	25.5 m
Occupancy Category	II
Seismic Use Group	I
Seismic Design Category	E
Importance Factor, I	1.0
Seismic Weight (W)	5,931 kN/m^2
Seismic Load Resisting System	BRBF with moment-resisting beam-column connections
R	8 (FEMA 450)
C_d	5 (FEMA 450)
C_t	0.03 (FEMA 450)
x	0.75 (FEMA 450)
C_u	1.4

2.7 Seismic Force Computation:

<u>Fundamental Period:</u>

Period, Ta: $T_a = C_t\, h_n^x$ …(1)

not to exceed: $T = C_u.T_a$ …(2)

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

<u>Base Shear:</u>

Base Shear, V: $V = C_s.W$ …(3)

$$C_s = \frac{S_{DS}}{\frac{R}{I}} \quad \text{…(4)}$$

C_s should not exceed: $C_s = \frac{S_{D1}}{T \cdot \left(\frac{R}{I}\right)}$ …(5)

C_s should not be less than: $C_s=0.044.I.S_{DS}$ …(6)

C_s should not be less than (seismic design categories E & F): $C_s = \frac{0.5xS_1}{\frac{R}{I}}$ …(7)

<u>Story Force:</u>

Force at each level: $F_x = C_{vx}.V$ …(8)

$$C_{vx} = \frac{w_x \cdot h_x k}{\sum_{i=1}^{n} w_i \cdot h_i k} \quad \ldots(9)$$

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

<u>Story Shear:</u>

$$V_x = \sum_{i=x}^{n} F_i \quad \ldots(10)$$

Using the preceding formulas, we are able to compute:
$C_s = 0.128$
$C_s \leq 0.134$
$C_s \geq 0.045$
$C_s \geq 0.055$ (for seismic categories E and F)
Therefore, V = Cs.W = 0.128W = 761.4 kN/m^2. See Table 3 for seismic force distribution values.

Table 3. Seismic Force Distribution

Level	W_i (kN/m^2)	h_i (m)	W_i x h_i^k (kN-m)	C_{vx}	Story Force F_x (kN)	Story Shear V_x (kN)
Roof	687	25.5	29,413	0.217	165	
7th	874	22	31,530	0.232	177	165
6th	874	18.5	25,789	0.189	144	342
5th	874	15	20,220	0.148	113	486
4th	874	11.5	14,857	0.108	82	599
3th	874	8	9,752	0.070	53	681
2th	874	4.5	5,003	0.035	27	735
1th	-	-	-		-	761
Total	5,931		136,564	1.000	761	

2.8 Computer Model Description:

- For simplicity, there is no distinction between roof and floor live load. All live load is modelled as floor live load.
- For simplicity, live load is not reduced.
- 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 3 and that its height wise distribution does not cause yielding of the BRBs.
- Braces are modelled as pin-ended.
- 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 modelled as rigid.
- 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 modelled as fixed at their bases.

III. Results and Discussion:

3.1 Maximum Story Drift Plot:

The corresponding maximum story drift for inverted V braced type structure is shown in figure 3.1 and 3.2 and for chevron braced type structure is shown in figure 3.3 and 3.4.

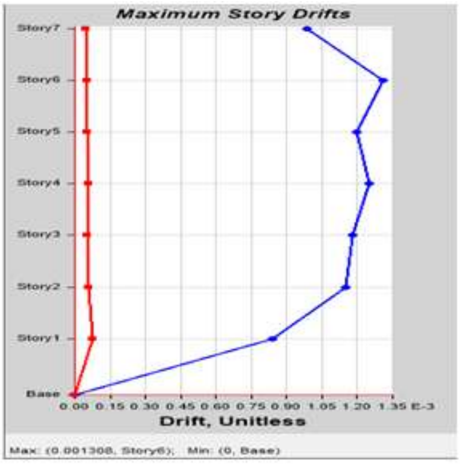

Figure 3.1. Max. Story drift- X direction

Figure 3.2. Max. Story drift- Y direction

Figure 3.3. Max. Story drift- X direction

Figure 3.4. Max. Story drift- Y direction

3.2 maximum Story displacement Plot:

The corresponding maximum story displacement for inverted V braced type structure is shown in figure 3.5 and 3.6 and for chevron braced type structure is shown in figure 3.7 and 3.8.

Figure 3.5. Max. Story Disp. - x direction

Figure 3.6. Max. Story disp. - Y direction

Figure 3.7. Max. Story disp. - X direction

Figure 3.8. Max. Story disp. - Y direction

IV. Conclusion:

- From the results it can be inferred that for the same plan depending on position of braces the performance of structure varies more.
- Based on the comparison between Chevron and Inverted V bracing, the drift and displacement for the inverted V bracing is more than Chevron bracing.
- Chevron bracing type will carry more than 40 % of lateral load to deform as inverted V bracing type.
- The connection for the BRBFs must be specially designed.

- Beam and Column must be designed for the extra axial load to carry which will act from the BRB.

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