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# Local Buckling of Buckling Restrained Braces

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**ABSTRACT :**

After extensive tests in the Center for Research on Earthquake Engineering (NCREE) and Taiwan University laboratories, buckling restrained braces (BRBs) have gained wide acceptance in seismic building applications in recent years. In order to facilitate cost-effective production, the cross-section optimization of the BRB is studied in this research. In order to slim down the BRB cross section, the role of the concrete strength in preventing the local buckling of the brace core is investigated using five full scale BRB specimens. Test results confirmed that the confinement requirements developed based on the lateral compressive stiffness of the infill concrete are not sufficient in the slimed BRB. Test results indicated that the wave length of the high mode buckling of brace core is about 2.25 times the width of core steel plate. Based on the flexural mode of the confining concrete under the lateral compression of core steel plate, it can be found that the required stiffness of the infill concrete is a function of the core steel plate width to confining concrete height ratio (*w/h*) and the core steel plate width to thickness ratio (*w/t*). It is found that if the brace core is made from ASTM A572 Grade 50 steel, the required concrete Young's modulus $E_c$ (in $kg/cm^2$) has to be greater that $18000(w/h)^3(w/t)$. The paper concludes with seismic design recommendations for the slimed BRBs.

**KEYWORDS:** buckling restrained brace, BRB, local buckling

## 1. INTRODUCTION

Most of BRBs are proprietary, but their concepts are essentially similar (Nakamura et al, 1998; Black et al, 2002; Tsai and Lai, 2002; Uang et al. 2003). BRBs commonly found are made from encasing a core steel cross-shape or plate member into a steel tube and infilled with the mortar or concrete. In order to reduce the friction from the brace compression, the unbonding material between the core steel member and the infill concrete is required. Thus, a BRB commonly consists of three components, steel core member, buckling restraining part and unbonding material (Fig.1). The steel core member is designed to resist the axial force from earthquake input energy with a full tension or compression yield capacity. The buckling restraining part is to prevent the steel core member's global flexural and local buckling failures. In recent years, the buckling restrained braced frame (BRBF) has been evolved into a very effective system for severe seismic applications (Tsai et al, 2008a; Tsai et al, 2008b). A number of new and retrofit building projects in Taiwan have selected BRBs as the response modification element to improve the seismic performance of the buildings. With these, the purpose of this research is to find the required stiffness or the strength of the infill concrete. This paper proposes a design criterion considering the core steel plate width ($w$), thickness ($t$), infill concrete height ($h$), and the concrete strength to facilitate the production of the slimed the BRB.

## 2. MECHANICAL PROPERTIES OF BRB

The BRB utilizes steel tube to prevent the global flexural buckling, and uses infill concrete to prevent the high mode local buckling.

### *2.1. Global flexural buckling*

In order to prevent the BRB from the global flexural buckling, the required stiffness of the steel casing can be computed from the following equation (Watanabe et al. 1988):

$$I_{sc} \geq \frac{P_y L_{sc}^2 \times (FS)}{\pi^2 E} \tag{2.1}$$

where $P_y=A_c F_y$ is the nominal yield strength of the brace and $L_{sc}$ is the steel tube length. To account for the starin hardening and the over-strength effects, a safety factor of 1.5 has been proposed. When the material over-strength and the strain hardening effects are known, it can be incorporated to replace the $P_y$.

### *2.2. Local buckling*

In order to develop the full BRB compressive capacity, there must be enough stiffness of the infill concrete. It was assumed that the steel tube is sufficiently rigid (Wada et al. 1994). The function of the concrete between the steel tube and the core member is like a number of elastic springs continuously distributed along the axis of the core member. Therefore, the $P_{cr}$ given in Eqn. 2.2 has been proposed as the failing capacity of the concrete. In order to prevent the local buckling of the brace core, the $P_{cr}$ needs to be greater than the peak BRB compressive strength. As shown in Fig.2.

$$P_{cr} = 2\sqrt{kE_i I_i} \tag{2.2}$$

$$k = \frac{E_c w}{h} \tag{2.3}$$

where $k$ is the spring constant per unit brace length. The $E_c$ is the concrete Young's modulus. The $E_i$ is the yielding Young's modulus and and $I_i$ is the moment of inertia about the core member weak axis. The spring constant given in Eqn. 2.3 suggests that the smaller the concrete wall thickness is, the greater the spring constant and the concrete failing capacity will be.

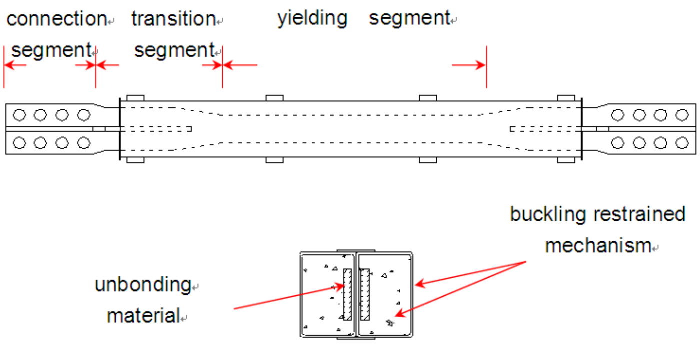



Figure 1. Details of the double-tube BRB member

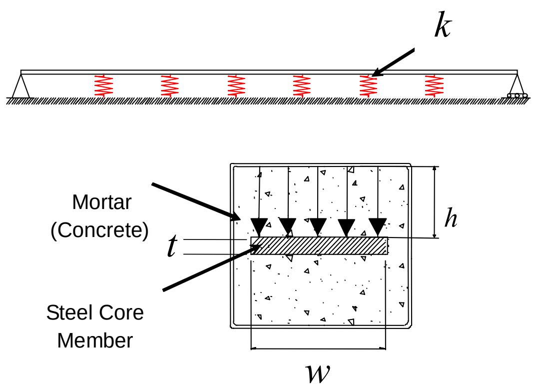



Figure 2. Illustration of effects of the concrete in restraining the brace core

## 3. PHASE I TESTS

### *3.1. Design of Phase I Specimens*

In order to verify the effectiveness of Eqn. 2.2 for the slimed BRB. Two similar BRB specimens were tested by using A572 Grade 50 steel and self-compact infill concrete in Phase I study. The cross section of the brace core is 100-mm by 18-mm. The loading protocol followed the guidelines (AISC 2005) recommended for BRBF. Without using the unbonding coating, a 1-mm and 2-mm thick gaps have been made between the brace core surfaces and the infill concrete (Fig. 3) for the Specimens U_G01 and U_G02, respectively. Both specimens satisfied the criterion given in Eqn. 2.2.

### *3.2. Key Test Results*

The actual strength of the core steel plate and the infill concrete are shown in Table 3.1. During the Phase I tests, premature local buckling failure occurred in both specimens (Fig. 4 and Photo 1). After removing the steel casing, the failure mode can be found in Photo 2. It appears that the Eqn 2.2 is not sufficient for the slimed BRB. In addition, the experimental responses indicated that the ratio of the buckling wave length to the brace core width is about a constant. For U_G01, the buckling wave length is about 225 mm as shown in Photo 2, while the wave it is about 230 mm in U_G02.

Table 3.1 Phase I material test results

| Phase I | Concrete strength | Steel strength | |
|---|---|---|---|
| Specimen | $f_c$’ (psi) | $F_y$ (Mpa) | $F_u$ (Mpa) |
| U_G01 | 5870 | 376 | 529 |
| U_G02 | 5083 | | |

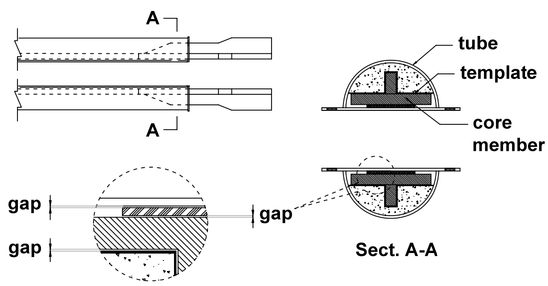



Figure 3 Details of the unbonding gap

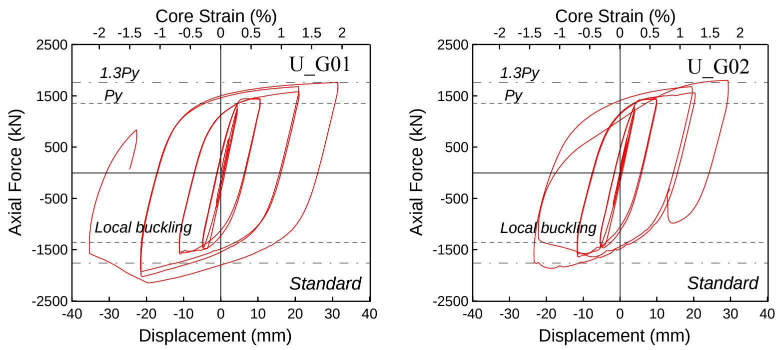
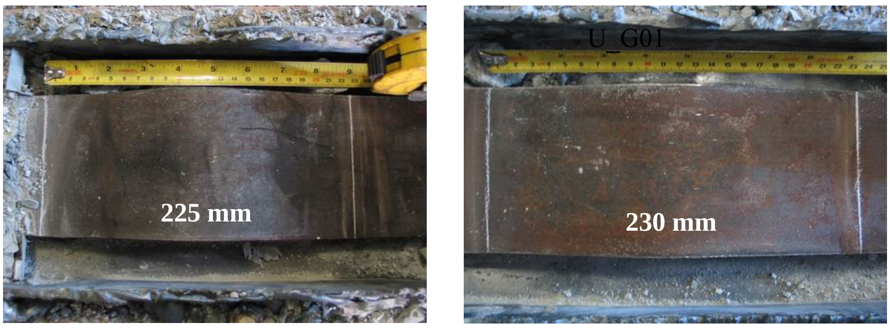



Figure 4 Cyclic responses of the two specimens in Phase I tests

Photo 1 U_G01 Specimen after test




(a) U_G01 (b) U_G02

Photo 2 Local buckling failure of Phase I specimens

### *3.3. The Demand of concrete strength*

Base on the experimental results observed in the Phase I tests, four assumptions are made as follows:

(1) The concrete failed in a flexural mode following the buckled core wave shape.
(2) The concrete was subjected to an uniformly distributed transverse load.
(3) The wave length of high mode buckling of brace is 2.25 times the core plate width.
(4) The yielding Young's modulus of the core steel plate is equal to 1% of the elastic Young's modulus.

*3.3.1 The bending stiffness of the concrete*

Assuming the concrete failed like a beam under the transverse load of $\omega$ as shown in Fig. 5. The effective lateral stiffness of the concrete can be approximated from the following equation:

$$\beta_c = k \cdot (2.25w) = \frac{\omega(2.25w)}{\dfrac{5\omega(2.25w)^4}{384E_cI_c}} = 6.74\frac{E_cI_c}{w^3} \tag{3.1}$$

$$E_c = 15000\sqrt{f_c'} \tag{3.2}$$

where $w$ is the width of the core steel plate. Eqn. 3.2 is the relationship between the concrete strength $f'_c$ and the the Young's modulus $E_c$ adopted in this study. The concrete moment of inertia can be computed from $I_c = wh^3/12$, in which $h$ is the concrete thickness between the core plate and the steel tube.

*3.3.2 The required stiffness of concrete*

When the steel core is made from ASTM A572 Grade 50 steel, the maximum axial load is:

$$P_{\max} = \Omega \cdot \Omega_h \cdot \beta_f \cdot P_y = 1.1 \cdot 1.3 \cdot 1.1 \cdot P_y \tag{3.3}$$

where $\Omega$ and $\Omega_h$ take into account the possible material over-strength and strain-hardening factors of the steel core member, respectively. In addition, the bonding factor $\beta_f$ represents the imperfect unbonding, the fact that the peak compressive is somewhat greater than the peak tensile strength during the large deformation cycles. In order to prevent local buckling failure, $P_{cr}$ in Eqn 2.2 must be greater than the $P_{max}$ in Eqn. 3.3. Thus, the required spring constant $k$ per unit brace length can be express as:

$$k \geq 7.42\frac{F_y^2}{E_i} \cdot (\frac{w}{t}) \tag{3.4}$$

The required effective lateral elastic stiffness of the concrete within the brace length of 2.25$w$ is:

$$\beta_c \geq 10100(\frac{w}{t}) \cdot w \tag{3.5}$$

*3.3.3 Recommendations for the required stiffness of the concrete*

From Eqn 3.1 to 3.5, if the steel core is made from ASTM A572 Grade 50 steel, the required concrete Young's modulus $E_c$ (in $kg/cm^2$) can be computed:

$$E_c \geq 18000(\frac{w}{h})^3 \cdot (\frac{w}{t}) \text{ or } f_c' \geq 1.44(\frac{w}{h})^6 \cdot (\frac{w}{t})^2 \tag{3.6}$$

Applying Eqn. 3.2, Eqn 3.6 suggests that the required concrete Young's modulus (or concrete strength) is a function of the core plate width to concrete thickness ratio ($w/h$), and the core plate width to thickness ratio ($w/t$). If the material over-strength of 1.5 and strain-hardening factors of 1.5 are assumed for ASTM A36 steel, the required concrete Young's modulus $E_c$ (in $kg/cm^2$) is:

$$E_c \geq 23000(\frac{w}{h})^3 \cdot (\frac{w}{t}) \text{ or } f_c' \geq 2.35(\frac{w}{h})^6 \cdot (\frac{w}{t})^2 \tag{3.7}$$

From Eqns. 3.6 and 3.7, it can be found that the smaller the concrete thickness, the greater the concrete strength is plate, the larger the concrete thickness to core plate width ratio must be. Fig. 6 provides the relationships among these parameters for seismic design of the slimed BRB. In this research, the cross sectional data of the BRB members (Table 3.2) used in several projects in Taiwan is studied. Table 3.2 also shows the required concrete strengths computed from Eqns. 3.6 and 3.7. These sample BRBs were tested to fracture before the actual fabrication of the BRBs for the construction sites. All sample BRBs used 8000 psi concrete. All specimens developed the cyclic tension and compression capacities of the BRBs without any premature local buckling failure. From the table, it appears that Eqns. 3.6 and 3.7 may be suitable for computing the minimum concrete thickness for the given concrete strength.

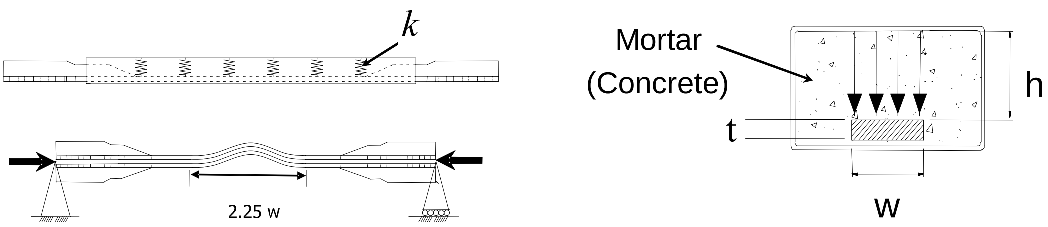



Figure 5 Schematic of concrete subjected to transverse loading during high mode buckling of the core plate

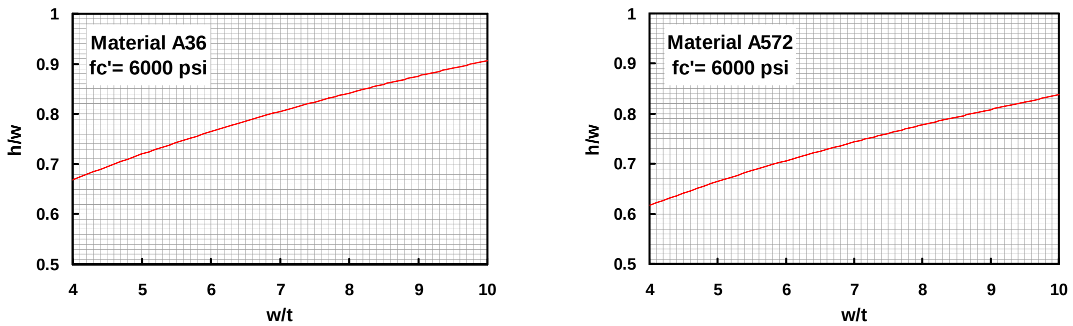



Figure 6 The required concrete height to steel plate width ratio (*h* /*w*) versus the plate width-to-thickness ratio (*w*/*t*) relationships

Table 3.2 Required concrete strength computed for various BRBs in some example buildings

| Case | material | *w*(mm) | *t*(mm) | *h*(mm) | $f_c'{}_{req}$ (psi) |
|---|---|---|---|---|---|
| Hospital | A36 | 180 | 28 | 142 | 5600 |
| Medical building | A36 | 110 | 36 | 80 | 2100 |
| Hightech factory building I | A572 Gr.50 | 95 | 16 | 64 | 7700 |
| Hightech factory building II | A572 Gr.50 | 220 | 40 | 167 | 3200 |
| University gymnasium | A572 Gr.50 | 230 | 24 | 182 | 7600 |
| Office building | A572 Gr.50 | 200 | 22 | 156 | 7400 |

Note：*w* and *t* are the core plate width and thickness, respectively
*h* is the concrete thickness between the core plate and the steel tube
*fc'* is the concrete strength

## 4. PHASE II TESTS

### *4.1. Design of Phase II Specimens.*

In order to verify the effectiveness of Eqn. 3.6 for the slimed BRB. A total of three BRB specimens were tested by using A572 Grade 50 steel and self-compact infill concrete in Phase II study. Without using the unbonding coating, a 2-mm gap has been made between the brace core surface and the infill concrete (Fig. 3). The specimens are designed based on Eqn. 3.6. The 18-mm thick core plate and 6000-psi concrete were used. Two parameters, core plate width and concrete thickness, varied in the Phased II tests. Three specimens, W90H60, W100H70 and

W100H60 (Table 4.1) are named. The numbers following the W and H stand for the steel plate width and the concrete height between the core steel plate and the steel tube, respectively.

***4.2. Key Test Results.***

The actual strength of the core steel plate and the infill concrete are shown in Table 4.2. During the Phase II tests, premature local buckling failure occurred in these three specimens (Photo 4 and Fig. 7). After opening up the steel casing, the local buckling failure modes can be found in Photo 5. After removing the brace core, the crushing failure of the concrete can be found in Photo 6. Fig. 7a indicates that premature compressive failure occurred during the last-second cycle in Specimen W90H60. The failure seems to occur in a strain of -0.75%, and the corresponding compression force was 1850 kN. Therefore, using this compressive load of 1850 kN to replace the $P_{cr}$ shown in Eqn. 2.2, then the required spring constant $k$ can be computed. Then using Eqns. 3.1 and 3.2, the minimum concrete strength, to resist the 1850 kN compressive brace force, can be found as 7200 psi. This value is very close to the concrete cylinder test result of 7748 psi given in Table 4.2, suggests that Eqn. 3.6 may be suitable for checking the infill concrete strength requirement. For specimen W100H70, the evident buckling failure displayed in Photo 5 can be recognized in Fig .7b. Because of the decreased axial force observed in the last-fourth cycle, the local buckling failure should have occurred during the last-fourth or last-third cycle. Therefore, assume the local buckling failure load is equal to the compressive force of 2050 kN developed in the end of the last-fourth cycle (Fig. 7b). Similarly, the minimum concrete strength, to resist the 2050 kN compressive brace force, can be found as 6300 psi. This value is somewhat less than the concrete cylinder test result of 8233 psi given in Table 4.2. Finally, Fig. 7c indicates that premature compressive failure occurred during the last-fourth cycle in Specimen W100H60. The failure seems to occur in a strain of -0.55%, and the corresponding compression force was 1700 kN. Similarly, the minimum concrete strength computed, to resist the 1700 kN compressive brace force, can be found as 7800 psi. This value is very close to the concrete cylinder test result of 7912 psi given in Table 4.2, suggests that Eqn. 3.6 suitable for checking the infill concrete strength requirement again.

Table 4.1 Dimensions of core plate and required concrete strength of Phase II specimens

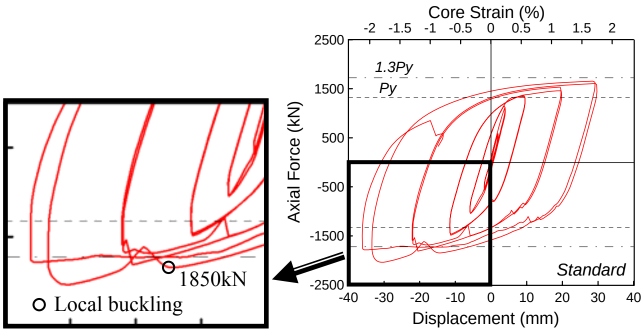
| Specimen | *w (mm)* | *h (mm)* | $f_c'_{req}$ (psi) |
|---|---|---|---|
| W90H60 | 90 | 60 | 5890 |
| W100H70 | 100 | 70 | 5427 |
| W100H60 | 100 | 60 | 13684 |

Table 4.2 Phase II material test results

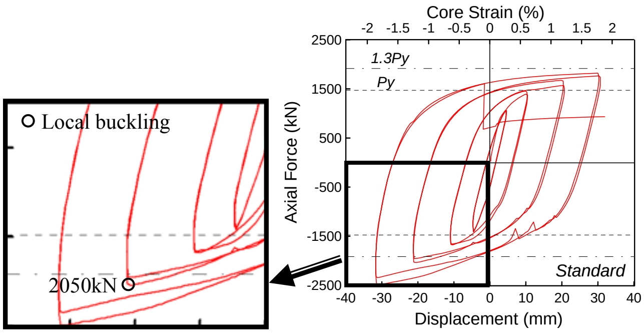
| Phase II | Concrete strength | Steel strength | |
|---|---|---|---|
| Specimen | $f_c'$ (psi) | $F_y$ (Mpa) | $F_u$ (Mpa) |
| W90H60 | 7748 | 409 | 527 |
| W100H70 | 8233 | | |
| W100H60 | 7912 | | |

Photo 4 Specimen W90H60 after test

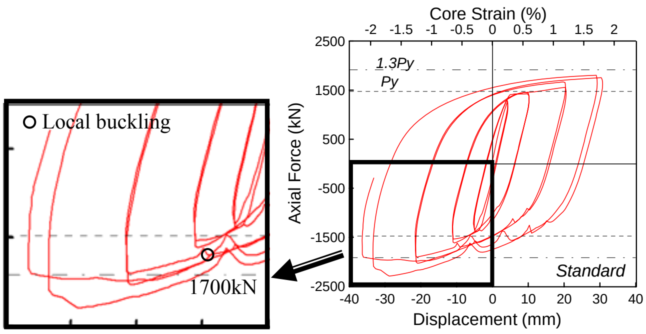

Photo 5 High mode buckling of braces




(a) W90H60




(b) W100H70




(c) W100H60

Figure 7 Test results and amplified hysteresis loops of the of Phase II tests

(a) W90H60

(b) W100H70

(c) W100H60

Photo 6 Concrete crushing after core plate local buckling failure of Phase II specimens

## 5. CONCLUSIONS

Base on the test results and the proposed design requirements for the infill concrete stiffness, conclusions and recommendations are made as follows:

- Test results show that the wave length of high mode buckling of brace is about 2.25 times the core plate width. Based on this observation, this paper proposes a method to compute the required concrete strength. It should be useful for the design of the slimed BRBs.
- This paper provides the relationships among the concrete strength, the width-to-thickness ratio of the core plate and the concrete thickness to core plate width ratio for seismic design of the slimed BRB. It is found that the smaller the concrete thickness, the greater the concrete strength is required. When the concrete strength is given, the larger the width-to-thickness ratio of the core plate, the larger the concrete thickness to core plate width ratio must be.

- This proposed method neglects the stiffness of the steel tube in preventing the local buckling of the core plate. In addition, the relationship between the concrete strength and the Young's modulus adopted in the proposed method is empirical. It may not be entirely suitable for full ranges of high strength concrete or mortar adopted in the fabrication of the BRB specimens and the samples for the building construction projects presented in this paper. Further research is needed in order to accurately compute the lateral stiffness requirements for preventing the local buckling of the BRBs.

## REFERENCES

AISC 341-05 (2005). Seismic Provisions for Structural Steel Buildings. American Institute of Steel Construction, Chicago, U.S.

Black, C.J., Makris, N. and Aiken, I.D. (2002). Component Testing, Stability Analysis and Characterizational of Buckling Restrained Braced Braces. Report No. PEER 2002/08, University of California, Berkeley.

Nakamura, H., Maeda, Y., Sasaki, T., Wada, A., Takeuchi, T., Nakata, Y. and Iwata, M. (2000). Fatigue Properties of Practical-Scale Unbonded Braces. Nippon Steel Technical Report, No. 82, 51-57

Tsai, K.C. and Lai, J.W. (2002). A study of Buckling Restrained Brace Frames. *Structural Engineering*, **17:2**, 3-32 (in Chinese).

Tsai, K.C., Hsiao P.C., Wang, K.J., Weng, Y.T., Lin M.L., Lin, K.C., Chen, C.H., Lai, J.W. and Lin, S.L. (2008a). Pseudo-dynamic tests of a full-scale CFT/BRB frame－Part I：Specimen design, experiment and analysis. *Earthquake Engineering and Structural Dynamics* **37**, 1081-1098.

Tsai, K.C., Hsiao P.C., Wang, K.J., Weng, Y.T., Lin M.L., Lin, K.C., Chen, C.H., Lai, J.W. and Lin, S.L. (2008b). Pseudo-dynamic tests of a full-scale CFT/BRB frame－Part II：Seismic performance to buckling-restrained braces and connections. *Earthquake Engineering and Structural Dynamics* **37**, 1099-1115.

Uang, C.M., Nakashima, M. and Richards, P. (2003). Seismic Analysis and Design of Steel Braced Frames (BRBF draft).

Watanabe, A., Hitomi, Y., Saeki, E., Wada, A. and Fujimoto, M. (1988). Properties of Brace Encased in Buckling-Restraining Concrete and Steel Tube. *Proceedings of 9th World Conference on Earthquake Engineering* **5**, 719-724, Tokyo-Kyoto, Japan.

Wada, A., Sacki, E., Takeuchi, T. and Watanabe, A. (1994). Development of Unbonded Braces. Nippon Steel Corporation, Tokyo, Japan.