Research and Application of Buckling Restrained Braces in Taiwan

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ABSTRACT

In recent years, experimental and analytical researches have been conducted in NTU to investigate the effectiveness of various kinds of unbonding material, the brace-to-gusset connection details, analytical models and the design procedures for the buckling restrained braces (BRBs). Tests conducted on more than 50 specimens confirm that the proposed BRBs employing double steel cores encased in twin steel tubes and infill concrete can stably sustain severe inelastic cyclic axial strain reversals. The double-cored BRBs can be conveniently connected to the gusset plate as in the traditional double-tee brace to gusset plate connections. Experiments include cyclic tests of BRB components and large scale single bay V-shaped buckling restrained braced frames (BRBFs). Test results indicate that the 2 mm thick silicon rubber sheets are effective in minimizing the difference between the cyclic peak compressive and tensile forces of the BRBs. The three V-shaped BRBF tests confirm that the steel brace core strain demands can be satisfactorily predicted from the story drift demand by geometry and incorporating the inelastic core length ratio. Test results also reveal that at a large story drift, the tensile strain in the tension brace was always greater than the compressive strain in the compression brace. This paper also introduces fully detachable BRB designs, non-destructive evaluation techniques for damage detections of BRBs. Finally, the paper concludes with the applications of the proposed BRBs in seismic retrofit as well as new construction projects in Taiwan.

Keywords: Buckling restrained braces, unbonded braces, cyclic loading tests, cumulative plastic deformation
INTRODUCTION

Buckling restrained braced frame (BRBF) has been evolved into a very effective system for severe seismic applications (Watanabe et al. 1988). Buckling restrained braces (BRBs) or Unbonded Braces (UBs) commonly found are made from encasing a core steel cross-shape or flat bar member into a steel tube and confined by infill concrete (Fig. 1). The steel core member is designed to resists the axial forces with a full tension or compression yield capacity without the local or global flexural buckling failure. When the brace is subjected to compressions, an unbonding material placed between the core member and the infill concrete is required to reduce the friction. Thus, a BRB or an UB basically consists of three components, including steel core member, buckling restraining part and the unbonding material. Figure 2 illustrates the typical cross sections of the BRBs or UBs proposed by various researchers. Most of these BRBs are proprietary, but their concepts are essentially similar. It can be found that the cross section of the steel core member is usually bi-axially symmetric, can be a cruciform, an H or a flat bar shape. The buckling restraining part can be constructed from mortar filled in the tube, reinforced concrete, reinforced concrete covered with FRP or all-metallic steel tubes. It has been found that a single cored brace to the gusset connection is typically a butt joint using several splice plates and two set of connecting bolts as illustrated in figure 3. In order to reduce the length and the number of bolts in the brace-to-gusset connection, the double-cored buckling restrained braces (DCBRBs) as illustrated in figure 4 have been developed (Lai and Tsai 2001) and extensively tested (Lai and Tsai 2001, Huang and Tsai 2002, Weng and Tsai 2002, Lin and Tsai 2003) in National Taiwan University (NTU) and Taiwan National Center for Research on Earthquake Engineering (NCREE) in the past few years. The proposed BRB members can be conveniently connected to the gusset plate in the same manner as that in the traditional double-tee brace to gusset plate connections (Fig. 5). These researches include the investigation of the effective unbonding material (Lai and Tsai 2001), cyclic and fatigue performance of the DCBRBs employing mortar infilled in the double tubes for A36 (Lai and Tsai 2001, Huang and Tsai 2002) and A572 GR50 (Lin and Tsai 2003) steel cores. The design criteria of the brace end connections and the inter-connecting ties between the two tubes have also been established through experimental tests (Weng and Tsai 2002). Tests have also confirmed that the self-compact concrete, much more cost-effective than the cement mortar, is a satisfactory alternative to restrain the core steel member in the BRBs (Lin and Tsai 2003).
RESEARCH ON DOUBLE-CORED BRBS

Effects of Various Unbonding Material

In order to find out which kind of material possessing satisfactory unbonding effects, a total of ten BRBs (identical in core cross sectional area as shown in figure 6 but varying in the unbonding material) were tested under cyclic increasing displacement at NTU (Lai and Tsai 2001). Table 1 summarizes the unbonding material used for each specimen and the corresponding cyclic loading protocol. The standard one refers to the protocols similar to the one provided by SAC (1997). The test setup is given in Photo 1. The typical cyclic response of the specimen is given in figure 7. The test results of the ten specimens are summarized in figure 8.

For the purpose of discussion, the axial load difference $\Gamma$ is defined as:

$$\Gamma = \frac{(C_{\text{max}} - T_{\text{max}})}{T_{\text{max}}}$$

(1)

where $C_{\text{max}}$ and $T_{\text{max}}$ are the maximum compressive and tensile brace forces at a same absolute axial deformation level.
Theoretically speaking, after the core member is yielded, the Poisson ratio \( \nu = 0.5 \) can be applied in the following calculations. The volume of the yielding steel segment should remain constant, that is:

\[
A_0 \cdot L_0 = A \cdot L
\]  

(2)

where \( A_0 \) and \( L_0 \) correspond to the original core cross sectional area and length, respectively, while \( A \) and \( L \) correspond to those after the brace is deformed in either tension or compression. Therefore, it can be shown that the axial strain is:

\[
\varepsilon = 1 - \frac{L_0}{L} = 1 - \frac{A}{A_0} \quad \text{and} \quad A = A_0(1 - \varepsilon)
\]  

(3)

Thus, the ratio between the compressive and tensile brace forces for a given (absolute) strain level could be:

\[
\Gamma = \frac{(C_{\text{max}} - T_{\text{max}})}{T_{\text{max}}} = \frac{A_0(1 + \varepsilon) - A_0(1 - \varepsilon)}{A_0(1 - \varepsilon)} = \frac{2\varepsilon}{1 - \varepsilon} = 2\varepsilon
\]  

(4)

Equation 4 suggests that the \( \Gamma \) is about 4% for an \( \varepsilon = 2\% \). But the test results shows in figure 6 exhibit much higher \( \Gamma \) values (the maximum is about 30% for \( \varepsilon = 2\% \)). This should be due to the imperfect unbonding mechanism and a substantial friction developed between the steel core member and the buckling restraining part. It is found in figure 8 that the 2 mm thick silicon rubber sheet has the least axial load difference (about 10% at an axial strain of 2%) under the cyclic increasing displacements. Therefore, for the subsequent tests of the BRBs, unless it is otherwise noted, the 2 mm thick silicon rubber sheets have been adopted for the construction of most of the specimens. In figure 7, it should be noted that the strain hardening effects are evident, comparing with the tensile yield capacity computed from the tensile coupon strength.

**Key Mechanical Properties of the BRBs**

In order to properly confine the BRB’s inelastic deformations inside the restraining tube, the cross sectional area \( (A_c) \) of the energy dissipation core segment \( (L_c) \) is smaller than that of the end joint regions \( (L_j) \). Schematic configuration of a DCBRB in the frame is illustrated in figure 9, in which \( L_c \) and \( L_{wp} \) represent the core length and the work-point to work-point length, respectively. Between the end and the core segment, a transition region as illustrated in figure 10 can be devices. It is confirmed by tests (Huang and Tsai 2002) that the effective stiffness, \( K_e \) of the BRB considering the variation of cross sectional area along the length of the brace, can be accurately predicted by:

\[
K_e = \frac{E A_j A_i}{A_j A_i L_c + 2 A_j A_i L_j + 2 A_i A_j L_i}
\]  

(5)
The relationships between the brace overall strain ($\varepsilon_{wp}$) and the inter-story drift $\theta$ can be approximated as:

$$\varepsilon_{wp} = \theta \sin \frac{2\phi}{2}$$  \hspace{1cm} (6)

where $\phi$ is the angle between the brace and the horizontal beam as illustrated in figure 11. The strain-to-drift ratio versus the beam angle $\phi$ relationships given in Eq. 6 are plotted in figure 12. If the ratio of the core length and the work-point to work-point dimension is:

$$\alpha = \frac{L_c}{L_{wp}}$$  \hspace{1cm} (7)

Thus, assuming the strain outside the core segment is negligible, the BRB core inelastic upper bound strain $\varepsilon_c$ can be express as:

$$\varepsilon_c \leq \frac{\varepsilon_{wp}}{\alpha}$$  \hspace{1cm} (8)

From Eq. 6 through 8, it can be found that if the inter-story drift demand is 0.02 radian, then the peak core strain would be close to 0.02 for a BRB having a length aspect ratio $\alpha=0.5$ and oriented in a 45 degree angle. The cyclic responses of a typical DCBRB constructed with two A36 flat steel cores (Fig. 4) using the 2 mm silicon rubber unbonding sheets are shown in figure 13. It is evident in figure 13 that a strain hardening factor of about 1.5 is appropriate in estimating the peak tensile strength for A36 core member. In addition, it appears that the following equation should be applied in estimating the maximum compressive strength possibly developed in a BRB:

$$P_{\text{max}} = \Omega \cdot \Omega_h \cdot \beta \cdot P_y$$  \hspace{1cm} (9)

where $P_y = A_c F_y$ is the nominal yield strength of the core section, $\Omega$ and $\Omega_h$ take into account the possible material over-strength and strain hardening factors of the core steel, respectively. In addition, the bonding factor $\beta$ represents the imperfect unbonding, the fact that the peak compressive strength
is somewhat greater than the peak tensile strength observed during the large deformation cycles. Therefore, the required stiffness of the steel casing in order to prevent the BRB from a global flexural buckling is (Watanabe et al. 1988):

$$I_{\text{tube}} \geq FS \frac{P_{\text{max}}(kL)^2}{\pi^2 E}$$ (10)

It is noted from figure 13 that for a properly fabricated BRB, a bonding factor $\beta$ of at least 1.1 is appropriate if the peak cyclic core strain demand is no greater than 0.02. Figure 12 also indicates that the experimental responses of the BRB can be accurately represented by using the two-surface plastic hardening material model implemented for the truss element in a general purpose frame response analysis computer program (Tsai and Chang 2001). Figure 14 further confirms that the same specimen, after subjecting the SEAOC/AISC (2001) standard loading protocols, sustained a total of 262 cycles of large fatigue strain before fracture. As the flexural buckling of a single-cored BRB member under large compress strains could occur (Tsai et al. 2002) at a section where the steel tube terminates (Fig. 3), it is recommended that the following stability criterion be met for connection details shown in figure 3 and figure 5:

$$P_{kL} \geq P_{\text{trans}}$$ (11)

where $P_{\text{max}}$ is given in Eq. 9 and $EI_{\text{trans}}$ is the flexural stiffness of the core member at a section near the end of the steel tube. As noted earlier, the double-cored BRBs can be conveniently connected to the gusset plate as shown in figure 5. As a result, the connection length is reduced as only one set of connecting bolts is required at each brace end. The connection length can be further reduced if a welded detail directly attaching the three edges of each tee to the gusset plate is adopted. An application example of the welded brace end joint in a SRC building will be given later in this article.

**Slip Resistant Bolted Connection Details in Brace End Double Tee to Gusset Joints**

In 2002, six BRBs featuring double-plate core with double tee end details were tested at NTU (Weng and Tsai 2002) incorporating two different levels of roughness at the tee-to-gusset contacting surfaces. A set of cyclically increasing forces and displacement was applied to find out the slip load of the brace end bolted joints. Then two addition sets of similar cyclically increasing forces and displacements but reduced in magnitude subsequently were applied to confirm the cyclically degrading of the slip capacity. Test results suggest that using 1.5 times the strength of the bolt for design can prevent slipping of the connection under cyclic loading reversals. That is:

$$R_{\text{str}} = N_s \cdot N_b \cdot (1.5 \cdot F_v \cdot A_b)$$ (12)

where $R_{\text{str}}$ is the slip-resistant strength of the connection, $N_s$ is the number of slip surface, $N_b$ is number of bolts, $F_v$ is the nominal shear strength of the bolt, and $A_b$ is the nominal tension area of the bolt. This research also investigated the longitudinal and transverse strain distributions in the web of the tee at the brace-to-gusset joints and concluded with the design recommendations for the brace end connection details.

**Tube-to-Tube Tie Connection Designs for Double-Tubed BRBs**

Since the proposed double-cored BRB consists of two independent units of bracing, tie connections between two units can be continuous or properly spaced. These ties can be made by welded bars (tab plates) as illustrated in figure 15 or various possible bolted details as illustrated
in figure 16. It is found that by using the elastic stability theory (Timoshenko and Gere 1961), the required strength $P_{req}$ and stiffness $\beta_{id}$ of the tie connection can be derived (Weng and Tsai 2002):

$$P_{req} = \frac{3}{L_{tube}} \left( \frac{P_{\max}}{2} - P_{\sigma} \right) \left( \frac{B \cdot \sigma_y}{E} + e \right)$$  \hspace{1cm} (13)

$$\beta_{id} = \frac{9}{2} \frac{L_{tube}}{B} \left( \frac{P_{\max}}{2} - P_{\sigma} \right) \left( 1 + \frac{E \cdot e}{B \cdot \sigma_y} \right)$$  \hspace{1cm} (14)

where $L_{tube}$ is the length of the buckling restraining tube, $P_{\max}$ is the maximum axial force suggested in Eq. 9 for the DCBRB, $P_{\sigma}$ is the critical eccentric load of the single tube derived from Secant formula (Timoshenko and Gere 1961), $B$ is the width of short side of the rectangular tube, $E$ is Young’s modulus of steel, $\sigma$ is yield stress of steel tube, $e$ is the eccentricity of load measured from neutral axis of the single tube. For example, if the design strength $P_y$ of the DCBRB is equal to 1960 kN, and estimated maximum axial force $P_{\max} = 4850$ kN, then according to the stiffness requirements noted in Eq. 14, the size of each buckling restrained tube is 350×150×6 mm, with $e = 32$ mm, $L_{tube} = 4054$ mm, $B = 150$ mm, $P_{\sigma}$ can be calculated from the Second formula as 810 kN. Based on Eqs. 13 and 14, the required strength $P_{req}$ and stiffness $\beta_{id}$ of the tie connections at every $1/3$ of the tube length are 38 kN and 306 kN/mm, respectively. Tests confirm that the proposed strength and stiffness requirements developed for the tie connection elements between the twin tubes can be conveniently applied in the design and construction of the double-cored BRBs subjected to large inelastic strain reversals.

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**Table 2 Specimen schedule for BRB frame tests**

<table>
<thead>
<tr>
<th>Frame (1)</th>
<th>BRB Specimen (2)</th>
<th>Yield Strength (kN) (3)</th>
<th>Total Bolts (F10T 24mm) (4)</th>
<th>$\alpha$ (5)</th>
<th>Peak Drift (% rad) (6)</th>
<th>Cycles for Fatigue Test (7)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSYF</td>
<td>NSY880W</td>
<td>880</td>
<td>16</td>
<td>0.362</td>
<td>1.25</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>NSY590E</td>
<td>590</td>
<td>12</td>
<td>0.362</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SYMF</td>
<td>SYM735W</td>
<td>735</td>
<td>16</td>
<td>0.371</td>
<td>1.5</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>SYM735E</td>
<td>735</td>
<td>16</td>
<td>0.371</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SYMSCF</td>
<td>SYM735SCW</td>
<td>735</td>
<td>16</td>
<td>0.185</td>
<td>1.25</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>SYM735SCE</td>
<td>735</td>
<td>16</td>
<td>0.185</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>UB735C</td>
<td>735</td>
<td>16</td>
<td>0.371</td>
<td>1.5</td>
<td>until fracture</td>
</tr>
</tbody>
</table>

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**Fig. 15 Schematic of welded tube-to-tube tie connections**

**Fig. 16 Several possible tie connection details**
Tests on Single Story V-shaped BRB Frames

In order to assess the performance of the double-T to gusset connection details, three large scale single bay V-shaped buckling restrained braced frames (BRBFs) constructed with the proposed BRBs have been tested in NCREE (Huang and Tsai 2002). The objectives of the study include: 1) investigating the experimental and analytical responses of the single bay V-shaped BRBFs each constructed with two BRBs in three different length aspect ratios, 2) investigating the steel BRB core strain versus inter-story drift relationships, and 3) providing guidelines for the analysis and design of BRBF for severe seismic applications. The schedule of three frame specimen and one BRB component is given in Table 2. The profile of all the BRBs is shown in figure 4. The frame elevations of these specimens are given in figure 17 and 18. The experimental setup for the frame test is given in Photo 2, while the BRB to gusset connection details can be found in Photo 3. The loading protocol for the frame tests and the components are illustrated in figure 19 and 20, respectively.
The cyclic and the fatigue performance of the UB735 are shown in figure 13 and 14, respectively. The experimental cyclic force versus deformation responses of the BRB component exhibit stable energy dissipation characteristics. Figure 13 also shows that experimental BRB responses can be accurately predicted using an inelastic truss element incorporating the two-surface plasticity model implemented in a general purpose finite-element computer program (Chang and Tsai 2001). Figure 14 indicates that same specimen after going through the inelastic excursions shown in figure 13, sustained a total of 262 cycles of fatigue strain on the order of 0.0125 before fracture. Figure 17 shows that the arrangement of BRBs can be configured un-symmetrically in order to accommodate architectural function needs, such as door or window openings. In this manner, the cross sectional area and the length of the energy dissipation segments of a pair of braces can be specifically tailored to avoid the potential unbalanced vertical load resultants while reach yielding at pretty much the same time. As shown in figure 21 through 23, the cyclic or fatigue performances of the frame systems and the BRBs indicate the proposed BRB components and framing system possess extremely stable characteristics. In addition, as shown in figure 24 and 25, the inelastic finite element models accurately predicts the frames’ and the BRBs’ experimental lateral force versus story drift responses for all test frames (only shown for Specimen NSYF herein). The three V-shaped BRBF tests confirm in figure 26 that the steel brace core strain demands can be satisfactorily predicted from the story drift demands by geometry (Eq. 6) and incorporating the ratio of the work point-to-work point dimension to the inelastic core length (Eq. 7 and 8). Test results in figure 26 and 27 also reveal that at a large story drift, the tensile strain in the tension brace was always greater than the compressive strain in the compression brace. This phenomenon is more pronounced as the inter-story drifts increase. This somewhat suggests that the peak compression and tension forces have a tendency to self-equilibrate as story deformation increases and reach a reduced unbalanced vertical force components resisted by the horizontal beam member.
Figure 23. Fatigue performance of three BRB frames without failure

Figure 24. Experimental vs. analytical frame shear versus lateral drift responses of Specimen NSYF

Figure 25. Experimental and analytical total brace shear vs. story drift relationships in Specimen NSYF

Figure 26. Experimental versus predicted core strain versus inter-story drift relationships for three frames

Figure 27. Tensile strain vs. compressive strain at peak inter-story drift for Specimens SYMF and SYMSCF

Figure 28. Steel cores are confined in the steel tubes without exposing the energy dissipating segments

Figure 29. Schematic of welded all metallic BRBs
All Metallic and Detachable BRBs

As noted above, single- or double-cored BRBs properly made from cement mortar or concrete filled in the steel casing possess satisfactory seismic resistant characteristics. However, as shown in figure 28, it does not allow visual inspecting the damage state of the inner core after an earthquake. In order to simplify the fabrication of the BRBs made from using infill mortar and unbonding coating, the all-metallic welded BRB as shown in figure 29 or the fully detachable BRBs illustrated in figure 30 have been developed and extensively tested in NTU (Tsai and Lin 2003). The detachable features shown in figure 30 provide the possibility of disassembling the BRBs for inspection after an earthquake. In order to allow the extension and contraction of two ends of a BRB, a stopper shown in figure 31 to lock into the restraining concrete or other buckling restraining part has been adopted to prevent the buckling restrainer from slipping off. Instead of applying the unbonding coating, a 1 mm or 2 mm gap has been experimented between the surface of the inner core and external restraining tubes. A total of thirteen detachable BRB specimens using A572 GR50 steel plates were fabricated and tested using the standard test protocol given in figure 32. Test results indicate that a 1 mm or 2 mm gap between the inner core and tubes is most effective in minimizing the difference between the peak BRB compressive and axial tensile force responses as illustrated in figure 33. After applying a standard loading protocol shown in figure 32, the specimen was never fractured. The same specimen was then subjected to a constant fatigue strains until fractured. Test results suggest that the all metallic and detachable BRBs can stably sustain severe cyclic increasing and constant fatigue inelastic axial strain reversals. As the specified constant strain varied from specimen to specimen, therefore, the number of fatigues cycles versus the fatigue strain relationships can be constructed as shown in figure 34. In addition, the cumulative plastic deformation (in terms of $\Delta\alpha$) is constructed and given in figure 35. Tests confirm that a typical fully detachable BRB exhibit stable and excellent hysteretic behavior as any other BRBs or UBs (restrained by mortar and steel casing) tested before. It is noted that the strain hardening factor for a BRB made from A572 GR50 is about 1.25 for a peak core strain less than 0.025. In this series of tests, it is found that the self compact concrete (SCC) with a compress strength of 56 MPa is a cost-effective alternative to the cement mortar for the infill in a steel casing.
Damage Inspection and Non-Destructive Testing of BRBs

As the energy dissipation segment of a conventional BRB is generally covered by the steel casing, direct damage detection may not be so straightforward. In order to inspect the BRBs after severe services, non-destructive test (NDT) techniques have been applied on more than 20 BRBs before, during and after the load tests (Tsai and Lin 2003). During various stages of each one specimen’s load test, it is done by simultaneously hitting the BRB at one end and measuring the stress wave at the opposite end as illustrated in figure 36 and 37. The NDT tests were conducted by hitting and observing the variation of stress wave form measurements collected from the BRB specimens. Receiver No. 2 shown in figure 37 was needed to normalize the impact force before comparing the impact and the response at the two BRB ends, respectively. A typical set of stress wave form is shown in figure 38, where the hitting/measuring Steps 1 to 3 were conducted, respectively, when specimen was on the floor, after connecting to the universal testing machine and after applying the standard loading protocol. Thus, the wave forms shown in Steps 1 to 3 in figure 38 suggest that the BRB had not been fracture. Step 4 was conducted after the same specimen had been fractured under the fatigue strain, and Step 5 was done after the specimen was removed from the load frame and brought down to the floor. It is evident in figure 38 that the stress wave forms are much calm after the fatigue fracture test. These results seem to suggest that the proposed NDT technique can be a simple way to detect whether the inner core is fractured or not. In addition, it is found in this study that the pressure wave (P-wave) measurements are more satisfactory than those of the shear wave (S-wave) for the stress wave NDT of BRBs.
EXAMPLE APPLICATIONS OF DOUBLE-CORED BRBS IN TAIWAN

Several new and retrofit projects in Taiwan have selected the double-cored BRBs as the energy dissipation elements to improve the seismic performance of buildings. Shee-Hwa United World Tower is a 46-story office building in Tai-Chung, designed before 1999, being seismically upgraded to accommodate an increased seismic hazard level. A total of 80 pieces of double-cored BRBs are being installed in the two opposite perimeter bays along the longitudinal direction of the frame.

A 10-story gymnasium at Chinese Culture University in Taipei has been proposed and now is under construction. The lateral force resisting structural system consists of mega braced frames in the longitudinal direction and build-up truss moment resisting frames in the transverse direction as shown in Photo 5. A total of 96 pieces of double-cored BRBs are installed in the three-dimensional steel frame. In this project, the maximum length of the double-cored BRB is about 11 meter long and the A572 GR50 steel core plate size is 45 mm × 260 mm, each inside a concrete-filled 500mm × 250mm × 15mm built up steel box casing. The peak axial tensile and compressive strengths could reach almost 10,000 kN. In order to confirm the cyclic performance of the large size double-cored BRBs, four full scale BRB components have been tested at NCREE (see Photo 6) using a loading protocol similar to that given in figure 32. Test results given in figure 39 and 40 confirm that the full scale large size double-cored BRBs can sustain a large number of inelastic cyclic increasing and constant fatigue strain reversals before failure. In addition, the cumulative plastic deformation exceeds 140 $D_{py}$, well meets the SEAOC/AISC prescribed requirements (2001).

Photo 7 shows the construction details of a 14-story Tzu-Chi Culture Building in Taipei. A total of 96 pieces of double-cored BRBs are installed in the opposite perimeter bays along the transverse direction of this SRC structure. It is shown in Photo 7 that the welded details have been adopted for the brace end connections effectively reducing the size of the double tee to gusset plate joint.
CONCLUSIONS

Based on the test and analytical results, summary and conclusions are made as follows:

(1) Using 2 mm thick silicon rubber sheets as unbonding materials can have smaller axial load difference under cyclic loadings. It is also confirmed that the self compact concrete (SCC) is a cost-effective alternative to the cement mortar for the infill in a steel casing.

(2) Extensive tests show that the hysteretic behaviors of the proposed double-cored BRBs are very stable. The double-cored features also make the BRB end connections shortened, reducing the
total number of bolts required by 50%. Test results also confirmed that the proposed all-metal and detachable BRBs can stably sustain severe inelastic cyclic axial strain reversals before failure.

(3) It is confirmed that the 11 meters long full scale BRB specimens can sustain the cyclic increasing deformations with a remarkable performance. It also possesses a quite acceptable fatigue life.

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