SEISMIC RETROFITTING OF LOW-RISE NONDUCTILE REINFORCED CONCRETE BUILDINGS BY BUCKLING-RESTRAINED BRACES

E. Yooprasertchai[1] and P. Warnitchai[2]

[1] *Graduate Student, School of Engineering and Technology, Asian Institute of Technology, Bangkok, Thailand*
[2] *Associate Professor, School of Engineering and Technology, Asian Institute of Technology, Bangkok, Thailand*
Email: st106327@ait.ac.th, pennung@ait.ac.th

ABSTRACT:

An experimental study was carried out to examine the applicability of bucking-restrained braces (BRB) to seismic retrofit low-rise, gravity-load-designed, concrete frame buildings. In the first phase of the study, a simple but effective BRB made from encasing an unbonded flat bar section with grout fill in a hollow square-section steel tube was developed through a series of quasi-static cyclic loading tests. The original design of BRB test specimens was based on available design guidelines and past research reports. However, they were found to perform very poorly due to the occurrence of several unexpected failures in or near the end of yielding zone of BRB specimens. After correcting those details, the final BRB specimen was able to successfully yield in both axial tension and compression up to ductility ratio of approximately 16. In the second phase of the study, an assemblage of a reinforced concrete column with non-seismic detailing and a diagonally arranged BRB was tested under quasi-static cyclic load. The test results showed that the BRB effectively yielded at a lateral drift of about 0.3%, which is much lower than the drift at which the column reached the peak strength (about 1 %). The force-deformation of the assemblage displayed ductile stable hysteretic loops from small to large lateral drift levels, without any sign of degradation, until the complete failure of the column at the drift of 2.5%. The results clearly demonstrated the potential of BRB in seismic retrofitting of non-ductile, low-rise concrete frame buildings.

KEYWORDS:
buckling-restrained braces, seismic retrofitting, quasi-static cyclic loading test, nonductile buildings, energy dissipation.

1. INTRODUCTION

The cyclic behavior of steel braces subjected to reversed compressive and tensile force exhibits poor energy dissipation due to the buckling of the brace when the loading exceeds the buckling limit. If the buckling of steel brace is restrained and the same strength is ensured in tension as well as compression, the energy absorption of the brace will be markedly increased and the hysteretic property will be simplified (Fig. 1). Energy input by seismic load is also expected to be dissipated greatly by these elements called buckling-restrained braces (BRB) or unbonded braces. In addition, if they are damaged, the rehabilitation after the earthquake is simple, since these elements are designed to be replaceable. The construction of BRB is quite simple because it can be made from widely available construction materials.

The design of conventional steel braces is normally governed by the buckling limit of compressive force, and hence a rather stocky brace section is normally obtained. When such stocky braces are connected to the structural frame, a significant increase in the lateral stiffness as well as period shortening is resulted, causing a considerable increase in the seismic base shear demand. The increasing base shear demand might exceed the safe capacity of the foundations. In contrast, since BRB can fully yield in compression as well as tension, a smaller cross section than conventional steel braces can be made. Analytical and experimental researches on BRB have been extensively conducted in Japan, Taiwan, USA and other countries. Most of the applications of these energy dissipators, however, are limited primarily to steel frames (Lopez, 2001 and Lopez et al., 2004).

Typical low-rise reinforced concrete buildings in low-to-moderate seismic zones are principally designed to carry only gravity loads. They have many typical characteristics that make them susceptible to the effects of severe earthquake loading. Longitudinal reinforcement of columns, for example, is commonly lap-spliced just above the floor level. The provided lap length and confinement by transverse reinforcement often are deficient for ensuring that the reinforcement can develop and sustain the yield stress under earthquake loading. Transverse reinforcement of column have typically poor configuration and wide spacing, resulting in inadequate confinement of the longitudinal reinforcement and column core for demand related to axial load, flexure and shear. In addition, columns are generally weaker than adjacent beams, so that the inelastic deformation may be concentrated in the columns under severe earthquake loading. Another important problem of deficiency seismic details is the discontinuity of positive beam reinforcement with a short embedment length into the joint. When the structure is subjected to seismic load, the moment at the beam end is reversal. If the bond strength is not sufficient, the positive moment from this action makes the bottom bar pull out. Due to insufficient joint shear strength, poor behavior of monolithic beam-column joints is also observed in many buildings after earthquake events. Generally, the first floors of the buildings have open spaces, glass windows or light partition walls to make the commercial or parking areas. Heavy partition walls are immediately placed above the ground floor, considerably increasing mass and stiffness above the first story. The upper stories thus move almost together as a single rigid block, and the story drift in the first floor during the seismic event is much greater than those in other stories, resulting in soft-story sway mechanism. Damage due to soft story may also occur at inter-story level, if that level has strength capacity less than adjacent floors. The resulting of these inadequacies, ranging form severe damage to complete building collapse, is evident during the earthquake in the past. It is thus vital that reinforced concrete structures especially life-safety structures, not designed in accordance to modern seismic code, shall be retrofitted to sustain earthquake loading. It is often more economically feasible to retrofit vulnerable existing reinforced concrete structures than to completely replace them.

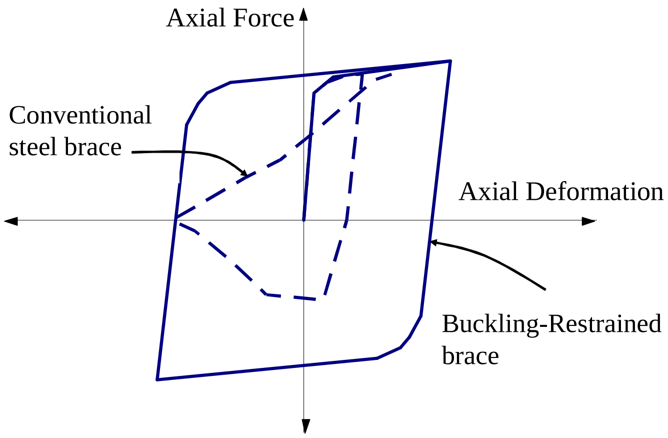


Figure 1 Cyclic behavior of conventional brace and buckling-restrained brace (adapted from Clark et al., 1999)

In this study, the basic idea to retrofit an existing building is to reduce the inelastic deformation demand in non-ductile building components (such as columns) by increasing the energy dissipation capacity of the building. Non-ductile RC columns typically can deform up to the lateral drift ratio of 1.0% without any sign of degradation. While a diagonally placed BRB in a building frame will reach the yield plateau at the lateral drift ratio as low as 0.2-0.3%. As a result, a significant amount of energy dissipation by BRB can be mobilized before the deformation limit of non-ductile RC columns is reached. In addition, if BRB is applied to a building with a soft or weak story, the strength and stiffness of the soft/weak story can be increased, which will reduce the concentration of inelastic deformation in that story.

In this paper, the applicability of BRB to retrofit non-ductile RC buildings is examined. A non-ductile RC column with a diagonally placed BRB was tested under quasi-static cyclic loading. The results clearly demonstrated the potential of BRB in seismic retrofitting of non-ductile, low-rise concrete frame buildings.

2. DESIGN OF TEST SPECIMENS

2.1 Design of BRB

BRB is designed based on the details described by Wada et al. (1988) and Clark et al. (1999). It is expected to withstand significant inelastic deformations when subjected to the forces resulting from the earthquake. The steel core should be designed to resist the entire axial load in the brace. The yield axial load, P_y, can be computed by Eqn. 2.1

$$P_y = F_y A_{sc} \tag{2.1}$$

where F_y = actual yield stress and A_{sc} = net area of steel core. The buckling of steel core will be restrained by grout mortar in the steel casing. The buckling strength of combination between grout mortar and steel casing, P_c, can be computed by Eqn. 2.2

$$P_c = \frac{\pi^2 EI_{cs}}{(kL)^2} \tag{2.2}$$

where kL is an effective length and EI_{cs} is the rigidity of the steel casing. The buckling strength of the casing must be greater than P_y to prohibit the buckling of BRB. It should be noted that the thickness of unbonded material needs to be sufficient large to allow the expansion of yielding core in compression. The transition section must be designed properly to ensure that inelastic deformation limited within steel core and elastic deformation occurs in other segments. It can be achieved by enlarging the section and welding the stiffeners in perpendicular direction. The longitudinal gap between the stiffen plates and filled mortar needs to be provided to accommodate the movement of yielding core.

2.2 Design of gusset plates

The gusset plate was designed to ensure that elastic deformation occurs. The required strength of gusset plate is computed based on the capacity of braces, adjusted by the overstrength factors, for example, compression overstrength, strain hardening, uncertainty of material, and fabrication tolerance. The gusset plates were designed according to the procedure described by Astaneh-Asl (1998). In order to prohibit the buckling of gusset plate and force the plastic hinge to occur within the allowable area, stiffen steel plate was welded to the gusset in the out-of-plane direction. Consequently, the brace can freely rotate at the end of bracing member (Astaneh-Asl at el., 1986).

2.3 Connection between BRB and concrete elements

Chemical anchor stud bolts were used for connecting BRB with concrete beams and columns. Typically, there are two main factors that must be considered in the connection design: tensile and shear force resistance. For tensile force resistance, three possible failure modes can appear in this load direction, namely, pull-out failure, concrete failure and tensile failure of the steel element depending on bolt spacing, concrete covering, bolt embedment, and so on. The maximum tensile load of steel failure depends on steel grades. The ultimate tensile resistance relies on the lowest resistance from the failure modes. For shear force resistance, two failure modes are governed, namely, concrete edge failure, for example, breaking away of the concrete component edge and the shear failure of the steel stud. The maximum and ultimate shear resistance of steel failure also depends on steel grades and lowest resistance from various failure modes, respectively.

3. TEST PROGRAM

The experiment was divided into two phases. In the first phase, only BRB was diagonally arranged in the steel column having the hinge at the bottom connected to the strong floor. Three specimens, namely, BRB1, BRB2 and BRB3 were examined the force-displacement relationship. Another phase is a sub-assemblage testing of both diagonal BRB and RC column to evaluate the compatibility between them.

3.1 BRB1 specimen

The brace core was made of 9×65 mm steel flat bar section and encased in a hollow structural section casing of 125×125×3.2 mm (Fig. 6). The length of yielding zone (L_c) was 2380 mm. The Ethylene Propylene Rubber (EPR) tape was wrapped the steel core until the unbonded material have approximately thickness of 1 mm to prevent the bond between steel and grout cement. The flowable nonshrink grout cement with ultimate compressive strength of 40 Mpa was filled in the casing. The restrained mechanism was designed such a way that global buckling of the brace can not be obtained. In order to limit yielding of steel core with in the casing, section at the brace ends were enlarged and welded with stiffening plates. The plates extended to inside casing were not only for stiffening, also for connecting the gusset plates by means of bolt connection. High strength bolt having diameter of 8 mm and splice plates were used for fixing between the brace and gusset plates. The bearing connection was used for this specimen. The longitudinal gap of 50 mm was provided to accommodate movement of the brace when subjected to compressive load. The polyurethane foam was filled in the gap to protect the direct bearing between enlarged section and grout cement. The configuration of BRB1 is shown in Fig. 2(a). One end of BRB specimens were diagonally arranged to the steel frame connected to the servo-hydraulic actuator (Fig. 4). Another was connected to the concrete base fixed to the strong floor by using gusset plate and chemical anchor stud bolts. To restrain the transverse movement, lateral braces were applied at the steel frame.

3.2 BRB2 specimen

The steel core section and steel tube section of BRB2 was the same as BRB1. The length of yielding region was 1804 mm. The length of restrained elastic section was extended to 345 mm. The bolted joint was designed as slip-critical joint condition. High strength bolts having diameter of 12 mm were used to connect between BRB and gusset plates. The same concrete base as BRB1 was used. Other elements which is not stated here had the detail as same as previous specimen. The configuration of BRB2 is shown in Fig. 2(b).

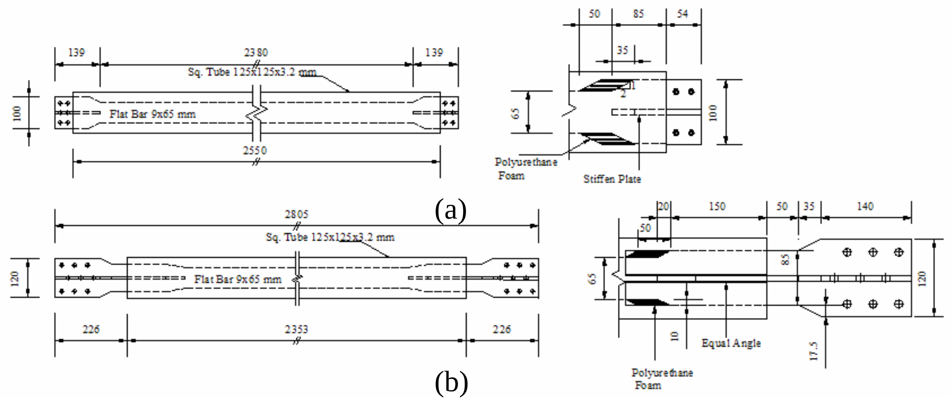
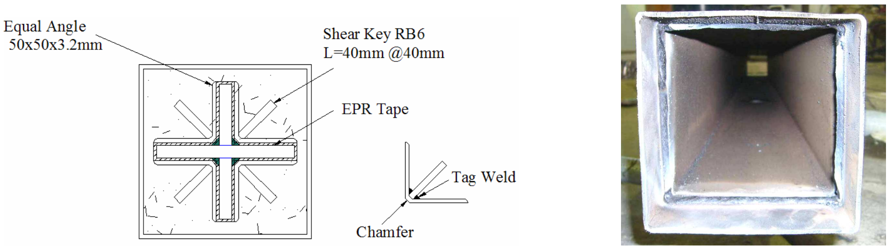


Figure 2 Configurations of (a) BRB1; (b) BRB 2 and BRB3

3.3 BRB3 specimen

The base fixed to strong floor was changed from concrete to steel. Welding was used to connect gusset plate and steel base together. The length of yielding zone (L_c) was 1995 mm. The equal steel angle of 50×50×3.2 mm with shear key was hence welded to form the slot allowing brace movement (Fig. 3a). The weld bead was also applied at 50 mm far from the both end of hollow casing to prevent the slippage between grout cement and steel casing (Fig. 3b).



(a) (b)

Figure 3 Configurations of (a) equal steel angle with shear key; (b) weld bead at the steel casing

3.4 CBRB specimen

The effective BRB was diagonally placed to the concrete column in the second phase. The BRB was bolted to gusset plates at its both ends. The gusset plate at the top end was connected to a short concrete beam extended from the column by means of chemical stud bolts. The column represents gravity-loaded-designed buildings previously tested by Worakanchana (2002). The loading point was however increased as twice in this study. The hysteretic behavior of column is shown in Fig. 10. Dimensions of column section are respectively 200 mm and 300 mm in perpendicular and parallel direction with applied force. Main longitudinal reinforcement of the column consisted of 24-DB 12. All longitudinal bars have lap spliced of 350-mm length at the position just above the base to reproduce the practical construction. Two 0.6-inch strands with the total force of 318 kN were applied to simulate axial force ratio ($P/f_c' A_g$, where P = axial load, f_c' = ultimate compressive strength and A_g = gross concrete section) of 0.3. The details of CBRB specimen are shown in Fig. 6.

3.5 Loading procedure

The testing was conducted in the Structural Engineering Laboratory at Asian Institute of Technology. The load was applied by using a MTS servo-hydraulic actuator with a capacity of ±500 kN. Loading history for BRB test specimens was adapted from *Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05* (Fig. 5).

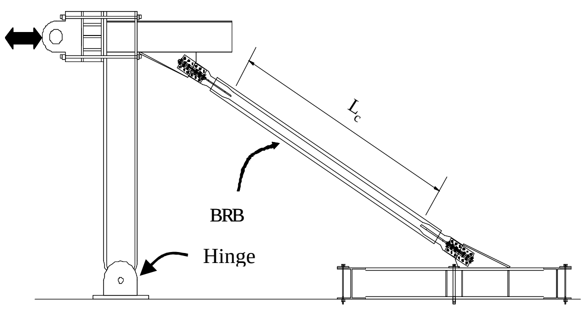


Figure 4 Test set up of BRB specimen

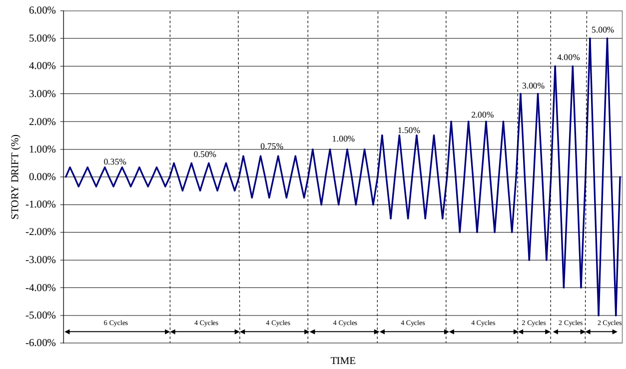


Figure 5 Loading History

4. EXPERIMENTAL RESULTS

4.1 BRB1 Specimen

The slippage of bolts connected between BRB and gusset plate was obviously clarified. When the joint slipped, the pinching was observed. It causes the unsymmetrical force-deformation curve when the brace was compressed (Fig. 7). The test was stopped at sixteenth cycles to repair the joint. All splicing plates were weld to restrain the joint movement. It was found that energy dissipation was greatly improved. The experiment was eventually stopped during the second cycle of 1.50% story drift (ε = 1.3%), which is approximately equal to ductility ratio of 6; due to the buckling of BRB at the bottom end of yielding section (Fig. 13a). The enlarged section was escaped from the casing and could not return to the casing resulting buckling of the brace.

4.2 BRB2 Specimen

The force-displacement relationship of BRB2 is shown in Fig. 8. It almost shows the elastic behavior both pulling (positive drift) and pushing (negative drift) movement at story drift of 0.35%. The brace was started to possess residual force when it back to original position at story drift of 0.50%. The maximum axial loads were equal to +174.7 kN for the pulling movement at the first cycle of 3.00% story drift and -187.3 kN for the pushing movement at the first cycle of 2.50% imposed story drift. The specimen buckle at the first cycle of 3.00% story drift (ε = 2.7%), which is approximately equal to ductility ratio of 13, at the top end of yielding region. The hysteretic loop in each cycle displays discontinuity at certain load due to slip of the concrete base connected to the strong floor. It is also observed that the grout cement was slipped from the steel casing (Fig. 13b). The restrained mechanism at the top end of steel core was hence vanished. The brace was easily buckled around the end of yielding zone. The accumulated energy dissipation was 143.2 kN-m.

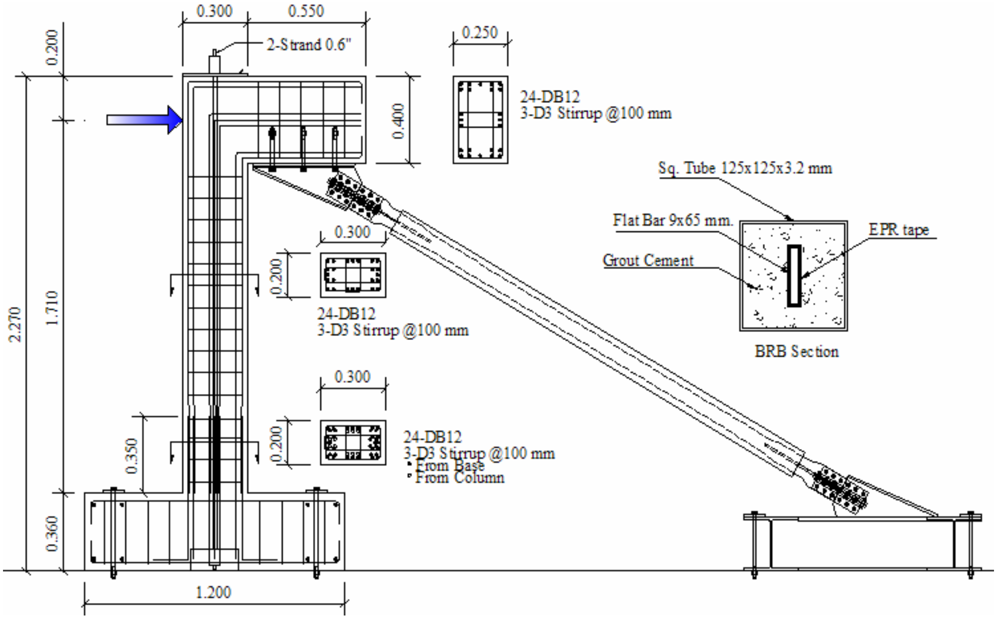


Figure 6 Configuration of BRB attached to RC column (CBRB)

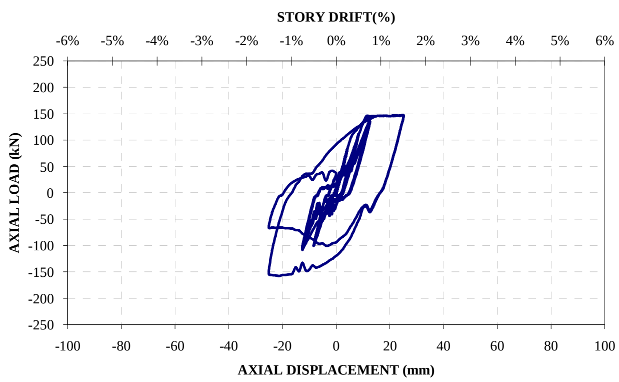


Figure 7 Force-displacement cyclic results of BRB1

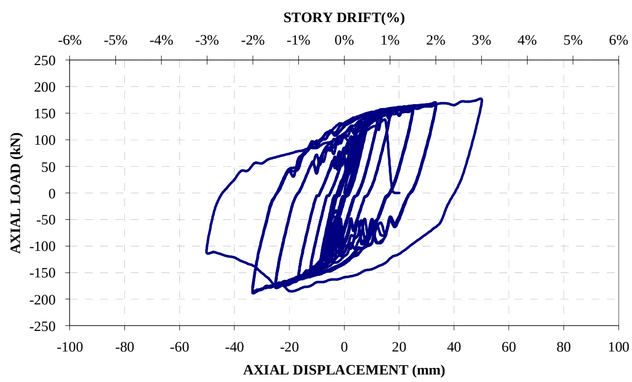


Figure 8 Force-displacement cyclic results of BRB2

4.3 BRB3 Specimen

Compared to BRB2, BRB3 could sustain inelastic deformation up to full cycles of 4.00% story drift (ε = 3.4%), which is approximately equal to ductility ratio of 16. The maximum axial loads were equal to +188.5 kN for the pulling and -240.5 kN for the pushing movement at the first cycle of 4.00% imposed story drift. It was shown that the maximum lateral load measured at peak negative drift was greater than that measured at peak positive drift. Maximum compressive overstrength factor was equal to 1.28. The accumulated energy dissipation was 283.9 kN-m. The force-displacement relationship of successful BRB is shown in Fig. 9.

4.4 CBRB Specimen

The force-displacement relationship of CBRB is shown in Fig. 11. The assemblage system shows a stable cyclic behavior both pulling and pushing direction. The story shear was slightly degraded for pulling direction. For pushing movement; however, no strength degradation was observed, since lateral force of BRB in the compressive direction is somewhat greater than that when BRB was pulled. The higher load in compressive side could compensate strength degradation of concrete column. The maximum story shears were equal to +245.1 kN for the pulling movement at the first cycle of 2.00% imposed story drift and -261.2 kN for the pushing movement at the first cycle of 2.50% imposed story drift. The accumulated energy dissipation of the system was found to be 114.3kN-m (Fig. 13). It was mainly contributed by BRB. Although BRB could increase strength, stiffness as well as energy dissipation on concrete column, its physical damage still occurred at the weak zone containing lap-splices. The minor flexural cracks were firstly observed at small drifts. It is then followed by flexural-shear crack. The cracks were increasingly propagated at higher drifts. The splitting cracks were clearly observed in the lap splice zone at the drift of 2%. An assemblage was finally failed at the second cycle of 2.5% story drift due to lap-splice failure (Fig. 13c). Chemical anchor stud connection was proved to sufficiently resist load transferred from BRB without any signs of failures.

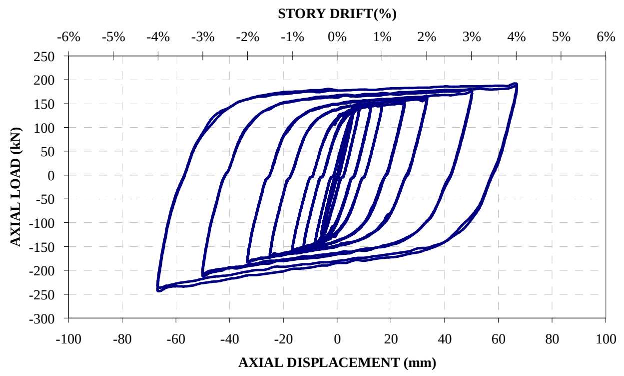


Figure 9 Force-displacement cyclic results of BRB3

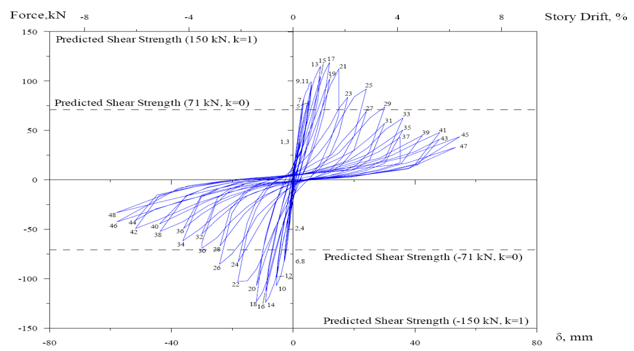


Figure 10 Force-displacement cyclic results of non-ductile concrete (Worakanchana, 2002)

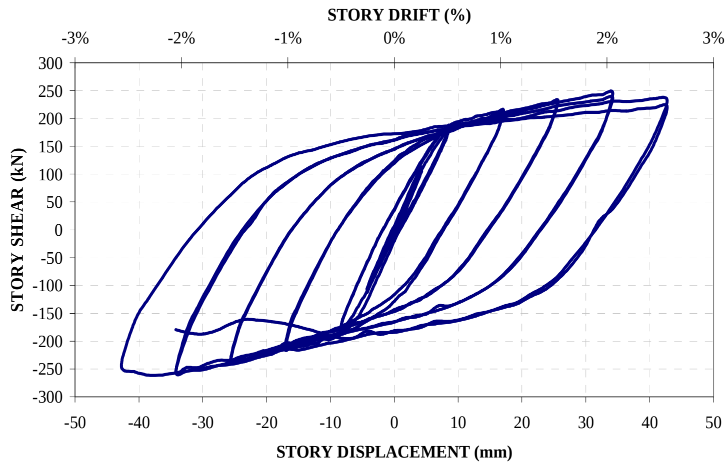


Figure 11 Force-displacement cyclic results of CBRB

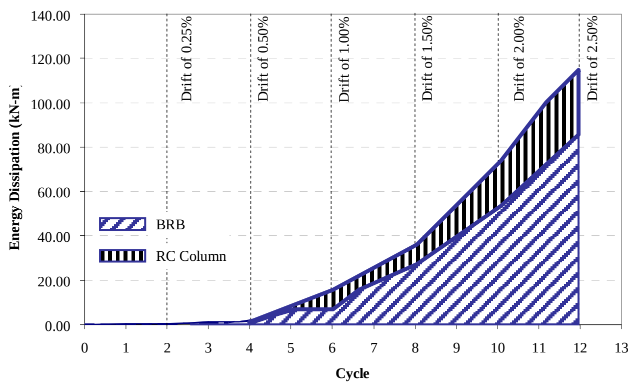


Figure 12 Accumulated energy dissipation of CBRB

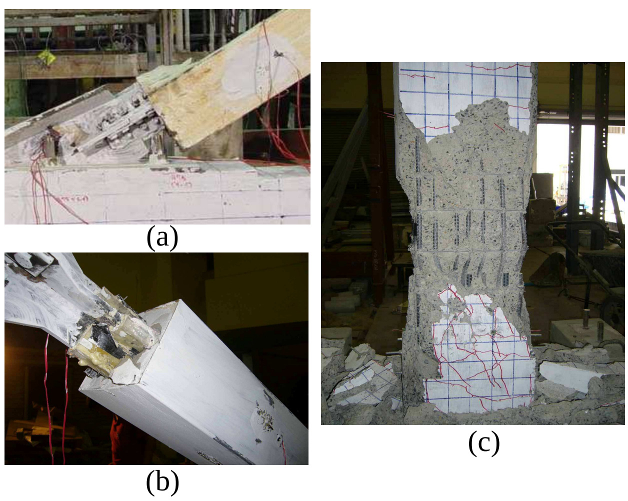


Figure 13 Failure of test specimen; (a) BRB1, (b) BRB2 and (c) CBRB

5. CONCLUSIONS

This paper summarized the experiment study of seismic retrofitting of low-rise, gravity-loaded-designed reinforced concrete buildings by means of BRB. Firstly, a study was summarized an improvement of BRB shown to be vulnerable at the transition section if it was not detailed properly. From BRB1, it was found that the slip-critical joint is essential to get fully yield in tension as well as compression. In addition, the enlarged section needs to be sufficiently embedded into the casing to protect the stiffeners running off the casing. The grout cement slipped from the casing was another problem found in BRB2. The restrained mechanism at the top end of the brace was hence vanished. The failures appeared to be critically sensitive to some detailing of BRB in such area. The failure of BRB occurred at end of plastic region due to the fact that no grout cement was applied in this section to avoid direct contact of the stiffening plates with the grout cement of the restraining part. Thus, the particular section has a lower restraining capacity than that of other sections. Using of weld bead and shear key demonstrated that the performance of BRB could be improved well. Presence of weld bead could significantly protect the slippage between grout cement and steel casing. While shear key applied on the steel angles could effectively provide the slot to allow movement of the brace. The BRB3 shows the stable force-deformation which it can yield in tension as well as compression up to story drift of 4% (ε = 3.4%), approximately equal to ductility ratio of 16.

An effective BRB diagonally assembled to RC column displayed greatly ductile stable cyclic loops from small to large lateral drift levels, with out any sign of degradation, until the complete failure of the column at the drift of 2.5%. Presence of BRB, not only increase strength and stiffness, also increase energy dissipation. BRB should be designed in such a way that it reaches yield plateau before the concrete frame does to reduce the seismic-induced damage in the structural element. The chemical anchor stud bolt connection proposed to connect between BRB and RC member satisfies to sustain the cyclic load. The test results demonstrated to be one of the possible procedures to retrofit the nonductile RC buildings.

REFERENCES

American Institute of Steel Construction, Inc. (2005). *Seismic Provisions for Structural Steel Buildings.* **ANSI/AISC 341-05**

Astaneh-Asl, A., Goel, S. C., and Hanson R. D. (1986). Earthquake-resistant Design of Double-angle Bracings. *American Institute of steel Construction Engineering Journal*, **Fourth Quarter**, pp 133-147

Astaneh-Asl, A. (1998). Seismic Behavior and Design of Gusset Plates. *Steel Tips Structural Steel Educational Council Technical Information & Product Service*

Clark, P., Aiken, L., Kasai, K., Ko, E., and Kimura, I. (1999). Design Procedures for Buildings Incorporating Hysteretic Damping Devices. *Proceedings, 68th Annual Convention, Santa Barbara, California SEAOC.*

Lopez, W. A., Gwie, D. S., Lauck, T. W., and Saunders C. M. (2004). Structural Design and Experimental Verification of a Buckling-Restrained Braced Frame System. *Engineering Journal, AISC*, ***Fourth Quarter***, pp 177-186.

Lopez, W. A. (2001). Design of Unbonded Braced Frames. *Proceedings, 70th Annual Convention, SEAOC.*

Wada, A., Watanabe, A., Hitomi, Y., Saeki, E., and Fujimoto, M. (1988). Properties of Brace Encased in Buckling-Restraining Concrete and Steel Tube. *Proc. 9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan*, **Vol. 4**, pp 719-724.

Worakanchana, K., 2002. Quasi Static Cyclic Loading Test of Reinforced Concrete Columns with Lap Splice in Typical Mid-Rise Buildings in Thailand. *Thesis No. ST*-02, Asian Institute of Technology.