A study towards the development of a low damage moment-resisting connections using blind bolts, CFCHS columns and a replaceable energy dissipating device

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ABSTRACT: The damage experienced in the Northridge and Kobe earthquakes has led to engineers having an increased awareness of the potential for brittle failure in welded connections. Prior to this, beam-column connections using welds were preferred over bolted connections because of the pinched hysteresis loops associated with the latter. Also, in the case where steel hollow sections are used as the columns, there is a lack access to the inside of the steel tube to tighten the nut if bolted connections are used. To cope with these problems a new type of bolt, the so-called blind bolt, which can be tightened from one side only, has been developed. This paper presents the results of FE studies into the behaviour of blind-bolted T-stub connections to concrete-filled steel CHS columns that are subjected to monotonic tensile and cyclic loading. A moment-resisting double T-stub type connection is proposed to connect steel universal beams with the concrete-filled CHS columns. Ultimately, it will incorporate an energy dissipating device at the bottom flange connection which will only allow movement under forces higher than those generated in the DBE. Numerical FE models have been used to predict the blind-bolted T-stub connection response and indicate that low damage connections should be achievable by adding an energy dissipation device at the bottom of the connection.

1 INTRODUCTION

A lot of brittle failures, especially at complete joint penetration (CJP) welds between the bottom flange of the beam and column flange, occurred at very low levels of plastic demand during the 1994 Northridge earthquake (FEMA-351 2000). The typical failure modes in the welded joint were summarized by Tremblay et al. (1995). Following this event, bolted connections have attracted the renewed attention of researchers because of their more reliable and ductile behaviour. Several improvements in the connections have been made (AISC 358 2010) such as that of triggering the plastic hinge in the beams away from the column face to dissipate energy. Though collapse prevention could be achieved using these connections, they would produce irrecoverable plastic deformation in which economic losses would be expected after an earthquake event.

Low-damage connections have been developed to reduce the economic losses post-disaster event. Those connections are as follows: post-tensioned steel tendon (PT) system (Christopoulos et al. 2002; Garlock et al. 2005), shape memory alloys (SMAs) (Sepúlveda et al. 2008; DesRoches et al. 2010), sliding hinge joint (SHJ) (Clifton 2005), self-centering SHJ (SCSHJ) (Khoo et al. 2012), and replaceable buckling restrained fuse (BRF) (Sarti et al. 2013). Clifton (2005) considered the PT joint to be impractical due to the effect of gap-opening, high compression force in the flange of the beam, and the required tendon length to remain elastic. Shape memory alloys in the austenitic state provide superelastic properties which have self centering capability. However, the behaviour of SMA depends on its thermal treatment which will be different for different size and material composition (Sepúlveda et al. 2008). SHJ has a point of rotation in the top flange plate that will minimize the slab damage. SHJ

also has asymmetric friction connection (AFC) that is rigid under the serviceability level earthquake (SLE) and starts sliding under the design level earthquake (DLE). Following the report made by Clifton (2005) which indicated that the MRSF may have excessive residual drifts following a major earthquake, the SCSHJ has been developed to increase the self-centering capability of SHJ by incorporating ring springs. To date, both the SHJ and SCSHJ have been used in beam-column connections to open section columns. A replaceable buckling restrained fuse (BRF) is another energy dissipating device that has been used in precast RC and timber structures (Sarti et al. 2013).

This paper proposes that replaceable BRFs be used at the bottom of the beam-column connection. However, it is possible that other low damage technologies may be used in future studies in conjunction with this blind bolted T-stub connection. Concrete filled circular hollow sections were used as columns as these sections provide high strength, stiffness, and ductility, in addition to aesthetic appearance. However, due to the lack of access to the inside of the steel tube to tighten the bolt, special bolts where the nut can be properly tightened from just one side, called blind bolts, will be used. There are several blind bolts available in the market such as the Huck high-strength blind bolt (Huck International Inc 1990), the Lindapter Hollo-bolt (Lindapter International 2012), Flow-drilling (France, Davison and Kirby 1999a), and the Ajax ONESIDE blind bolt (Ajax Engineered Fasteners 2002). The Ajax ONESIDE blind bolt will be utilized in this paper as they are representative of high strength blind bolts.

Taking the benefit of the concrete infill, an additional anchorage component has been added to the blind bolt to increase the strength and stiffness of the connection. Yao et al. (2008) have tested single and groups of cogged anchor blind bolts (CABBs) in CFCHSs (Figure 1a). The original intention was to make the CABB in one piece. However, due to the high cost needed to implement this intention and also the difficulty in achieving the required alignment of the cogged anchored bolt during tightening, headed anchor blind bolts (HABBs) have been proposed by Yao et al. (2011) as shown in Figure 1b. A similar concept using headed anchor has also been investigated by Pitrakkos and Tizani (2013), and Mahmood et al. (2014). Based on the previous work done by Oktavianus et al. (2014) into the behaviour of groups of HABBs in CFCHSs when subjected to monotonic tensile loading, it was shown that the inclusion of a through bolt in the group of HABBs (which will also be used when applied in practice) is very important to increase the stiffness and strength of the connection. Therefore, this paper will include the through bolt in the T-stub connection. A T-stub connection has been used because it can develop about 90% of the fixed-end moment (Kulak et al. 2001). The effect of the stem of the T-stub and the beam was neglected to simplify the FEA conducted in this study to model the group behaviour of HABBs due to cyclic loading. The behaviour of the connections including T-stub, blind bolts, CFCHS, and replaceable BRFs as the energy dissipation is presented. Moment-rotation relationships for both cases have been constructed and discussed.

(a)   (b)

Figure 1. (a) CABB (Yao et al. 2010); and (b) HABB (Yao et al. 2011).

2 CASE STUDIES

There are two building models used in this paper. The first building model (case A) was adapted from a case study presented by Clifton (2005) for a design example of a MRSF for Auckland. It was chosen as a representative of low seismicity regions. A 530 UB 82 beam connected to a 610 UB 101 column at the 9th storey of a 10 storey building was used in that study. The column has been modified according to this research purpose which uses CFCHS. CFCHS 508 x 12.7 with steel tube grade of C350L0 and with concrete compressive strength of 50 MPa have been chosen due to the similarity in effective stiffness. The beam and the column have yielding moment capacity equal to 543 and 1300 kNm, respectively. The inter-storey height was 3.5 m and the plan view is shown in Figure 2. The moment demand and tension force at top flange level (V^*_{tfb}) due to design based earthquake (DBE) were equal to 350 kNm and 574 kN, respectively.
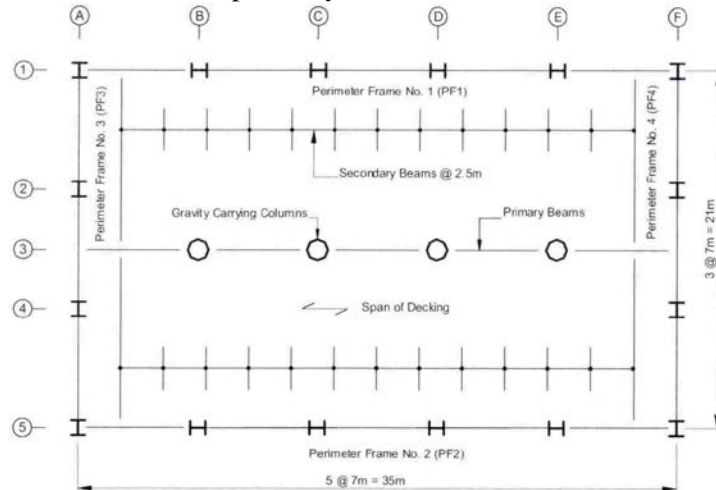

Figure 2. Plan view of case A (Clifton, 2005).

The second case study (case B) was adopted from Khoo (2013) which presented a design example for the SCSHJ in MRF in the Te Puni Village Tower Building, Wellington (which currently uses SHJ). It was chosen as a representative of high seismicity regions. A 460 UB 67.1 beam connected to a 530 UB 92.4 internal column at the 1st storey of a 10 storey building was used in that study. As in case A, the column in case B has also been modified into a CFCHS 457 x 9.5 with steel grade of C350L0 and with concrete compressive strength of 50 MPa. The beam and the column have yielding moment capacity equal to 490 and 850 kNm, respectively. The building plan view and layout is shown in Figure 3. The moment demand and tension force at the top flange level (V^*_{tfb}) due to the DBE were equal to 269 kNm and 536 kN, respectively. The rotational demand due to the maximum considered earthquake (MCE) was assumed to be equal to 40 mrad, which is reasonable for a very rare EQ.
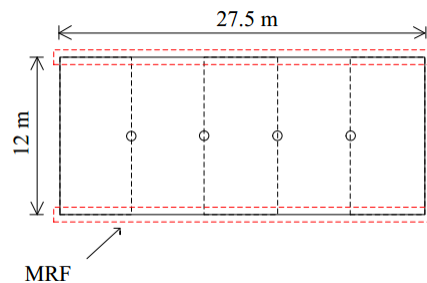

(a)
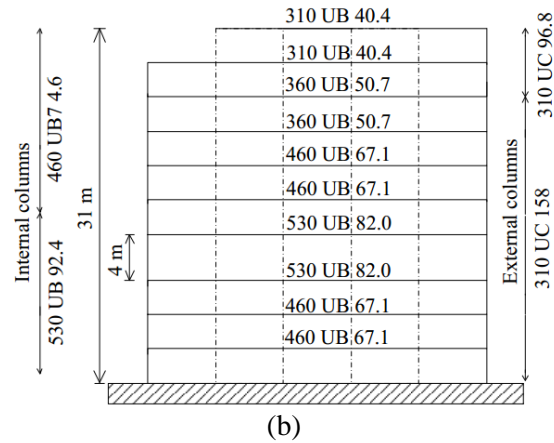
Figure 3. Case B: (a) building plan view (b) MRF layout and member sizes (Khoo 2013).

Figure 4 shows the connection layout for case B and the layout for case A is similar to this. Section D-D and E-E show the sections at the top and bottom bolt-lines of the T-stub, respectively. Since a CHS was being used, there was a geometry limitation that governs the embedment depth of the headed anchor. For case A: in the top part of the T-stub, there were 2M24 HABBs with embedment depths of 100 mm, and 1M30 through bolt in the middle. In the bottom part of the T-stub, there were 2M30 HABBs with embedment depths of 130 mm. For case B: in the top part of T-stub, there were 2M20 HABBs with embedment depth of 100 mm and 1M24 through bolt in the middle. In the bottom part of the T-stub, there were 2M24 HABBs with embedment depths of 120 mm. The thickness of the curved endplate of the T-stub has been chosen to minimise prying action, and it was equal to 30 mm and 25 mm for case A and B, respectively.

(a) Front view   (b) Section D-D   (c) Section E-E
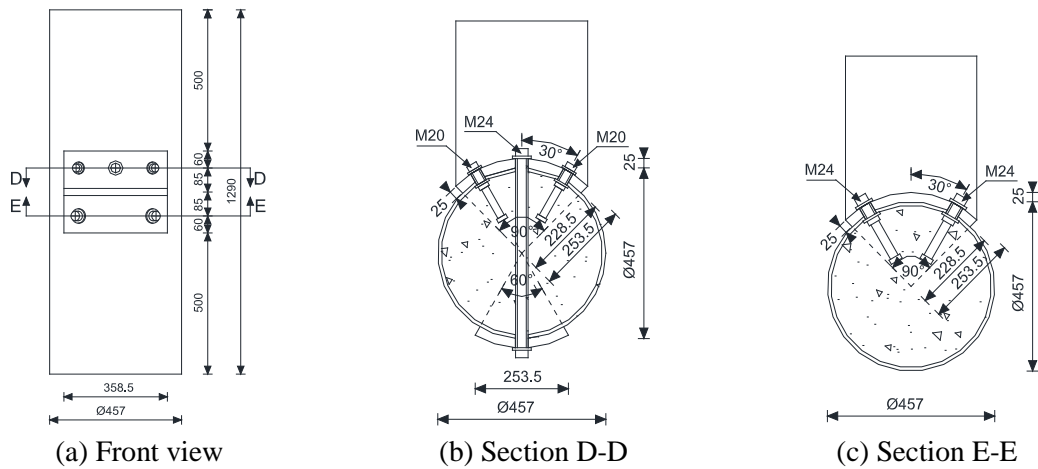
Figure 4. Connection layout for case B: (a) Front view; (b) Section D-D; (c) Section E-E.

3   ENERGY DISSIPATION DEVICE: REPLACEABLE BRF

An energy dissipation device, i.e. four replaceable steel BRFs, was used in both case A and case B. It was installed at the bottom of the connection as illustrated in Figure 5a. The same concept as that used for the SHJ in which the top corner of the connection is the point of rotation was used here. Furthermore, the cyclic behaviour of the bar was similar to the work done by Sarti et al. (2013). The layout and dimension of the replaceable BRF are shown in Figure 5c and Table 1, respectively. The elastic modulus, yield stress, and post-yield stiffness were equal to 200 GPa, 360 MPa, and 2 GPa, respectively.
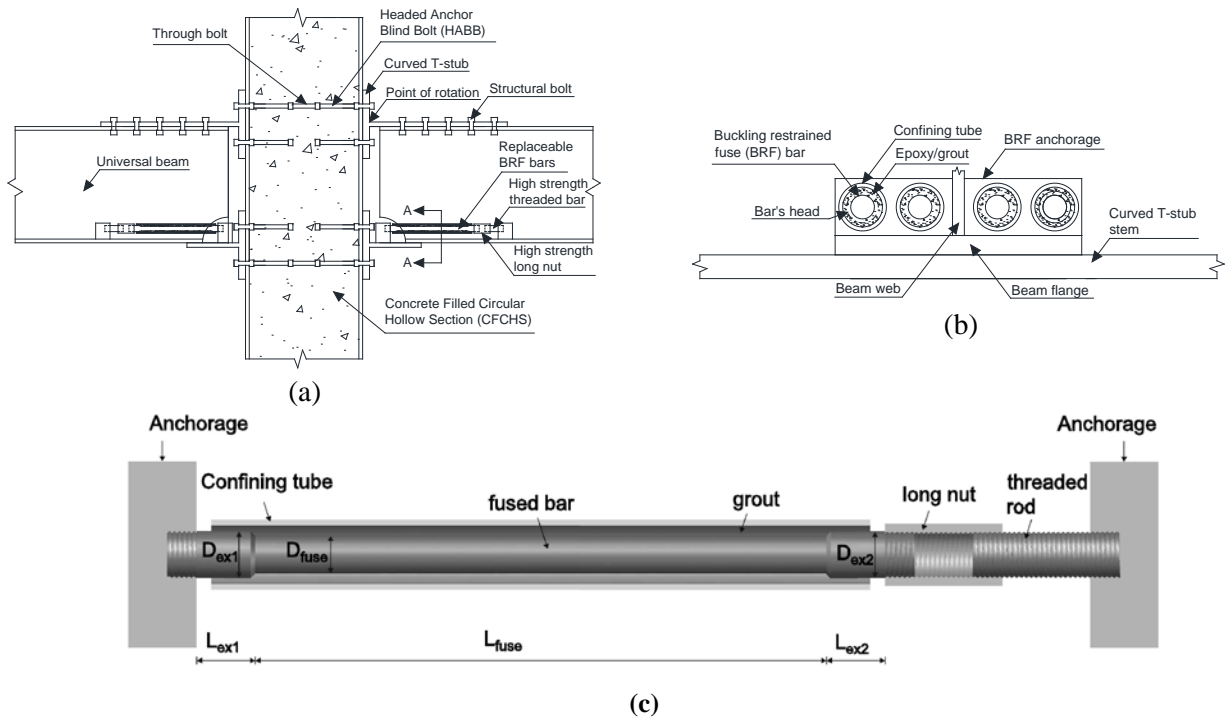
Figure 5. Blind bolted T-stub connection with replaceable BRF: (a) whole connection; and (b) section A-A (c) BRF layout.

Table 1. BRF dimension

Thread	D_{fuse} (mm)	L_{fuse} (mm)	$D_{ex1}=D_{ex2}$ (mm)	$L_{ex1}=L_{ex2}$ (mm)	Tube int. Diam. (mm)	Tube ext. Diam. (mm)
M30	26	390	32	40	40.3	48.3

The hysteresis behaviour of replaceable BRFs for "case A" and "case B" is shown in Figure 6. The device will remain elastic up to DBE and starts yielding as the demand exceeds the DBE. This is important to ensure the deformation in the building is small up to the DBE.
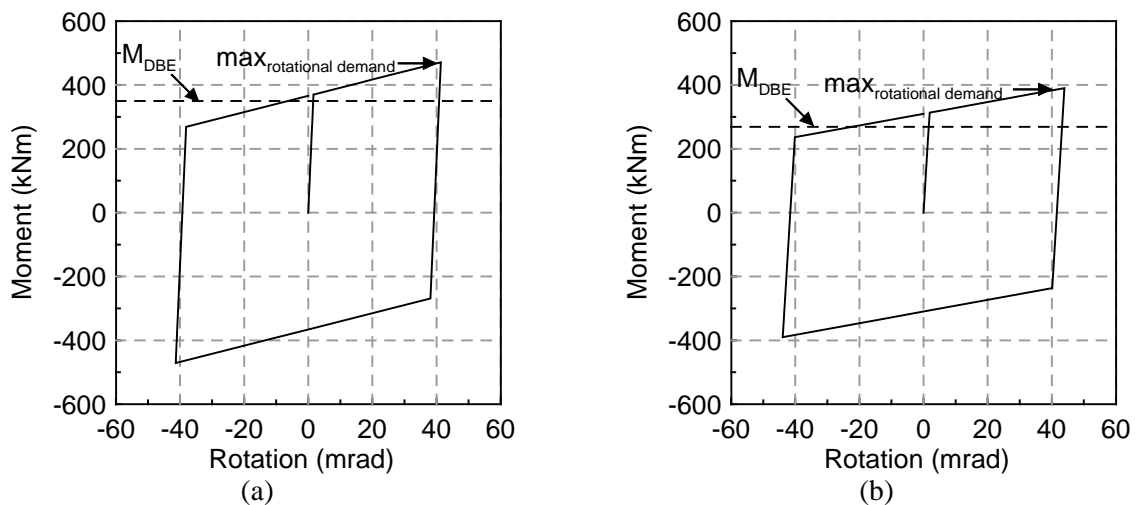

Figure 6. BRFs hysteresis behaviour: (a) case A; (b) case B.

4 FINITE ELEMENT ANALYSIS FOR T-STUB CONNECTION

4.1 Finite element modelling

Figure 7 shows a quarter model of "case A" using a finite element package called ABAQUS (2011). For simplification, the blind bolt and nut were modelled as one solid element, and the collapsible washer was also modelled as solid washer. Minimum pretension force approximately equal to 600 MPa was applied using temperature pressure to the middle of the shank of the blind bolt. Material properties for steel elements used in this research have been summarised in Table 2. A concrete compressive strength of 50 MPa was used and was modelled using the Mander model (Mander et al. 1988). Moreover, concrete damage plasticity (CDP) which was able to represent the concrete plastic behaviour was used. A concrete tensile strength equal to $0.56 \sqrt{f_c^{'}}$ was used (ACI 318M, 2011).

Fracture energy of 160 N/m was used for a concrete strength of 50 MPa following the ABAQUS documentation (ABAQUS 2011).
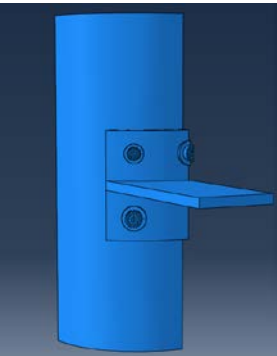

Figure 7. Quarter finite element model.

Table 2. Material properties for steel elements

Note	Steel tube	Curved endplate of T-stub	Blind bolt, nut, and washer
f_y or f_p * (MPa)	350	310	600
f_u *(MPa)	430	430	830

* f_y, f_p ,and f_u represent yield stress for steel, proof stress for bolt, and ultimate strength, respectively.

4.2 Finite element analysis result

Both monotonic and cyclic loading were applied to the specimens. A monotonic tensile force was applied using displacement control, whereas cyclic loading was applied using force control. The outward displacement was measured from the middle of the curved endplate adjacent to the T-stub stem. In a previous study (Agheslui 2014 & Oktavianus et al. 2014) mentioned that the maximum allowable force should be maintained to be less than 60% of the ultimate capacity of the system to ensure the rigidity of the connection (i.e. preventing the concrete cone to occur and blind bolt to yield). Therefore, a cyclic tensile force with maximum amplitude of approximately equal to 60 percent of the nominal ultimate capacity was applied to "case A" to observe the stiffness degradation of the connection. The behaviour of "case A" due to monotonic and cyclic loading was shown in Figure 8. After 5 cycles in tension (which is approximately the maximum number of cycles expected in a severe earthquake), the secant stiffness dropped by 5% and a residual displacement of 0.3 mm was observed. Even after an additional 15 cycles, the total secant stiffness dropped by 8.3% and a total residual displacement of 0.4 mm was attained. This indicated that there was negligible stiffness deterioration of the connection at a load level less than 60% of the ultimate capacity.
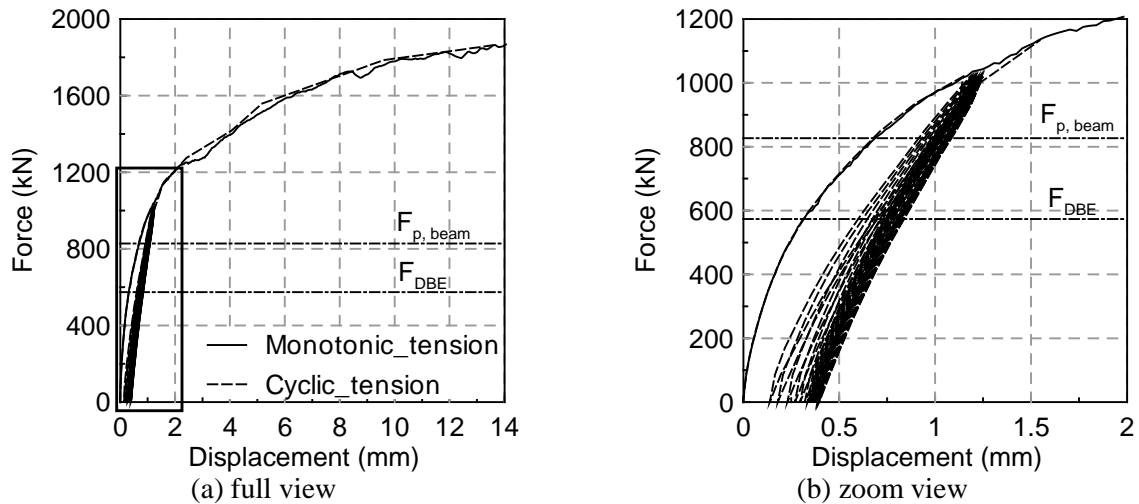
Figure 8. Behaviour of "case A" due to monotonic and cyclic in tension: (a) full view; (b) zoom view.

Refer to "case A", the force due to DBE was approximately equal to 36% of ultimate capacity, which was lower than 60% of ultimate capacity. Therefore, full cyclic loading (i.e. compression and tension) at 40% of ultimate capacity was applied to a T-stub blind bolted connection in case A. Ultimately, moment-rotation behaviour of the connection without energy dissipation device could be obtained as shown in Figure 9a. It was shown that the connection remained rigid and has negligible yielding even at a load level higher than DBE.

For case B, the force due to DBE was approximately equal to 48% of ultimate capacity, which was also lower than 60% of ultimate capacity. Therefore, full cyclic loading (i.e. compression and tension) at 50% of ultimate capacity was applied to a T-stub blind bolted connection in case B. Moment-rotation behaviour of the connection without energy dissipation device could be obtained as shown in Figure 9b. It was also shown that the connection was rigid in a braced frame and semi-rigid in unbraced frame and has negligible yielding at a load level up to DBE.
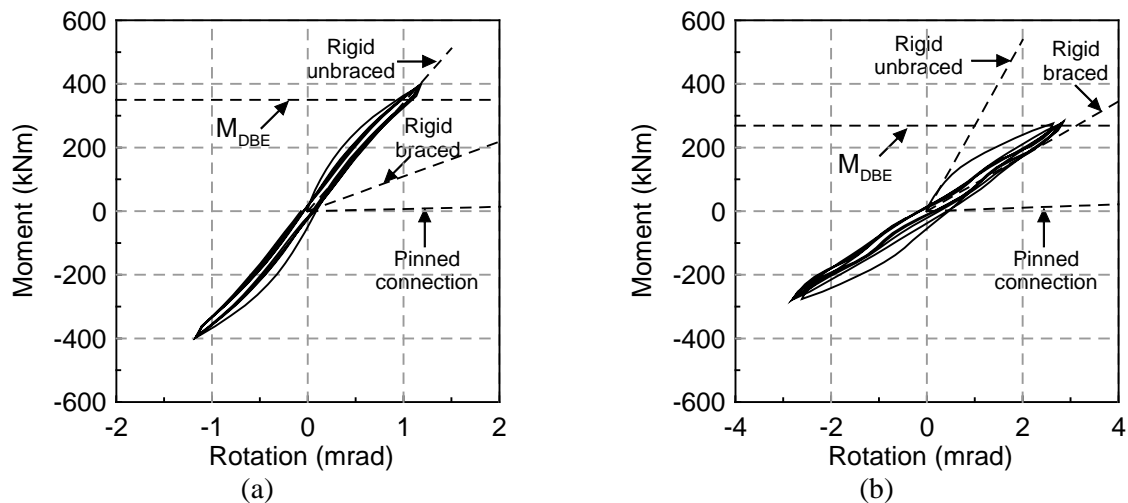

Figure 9. Moment-rotation behaviour for T-stub connection: (a) case A; (b) case B.

5 COMBINED HYSTERESIS LOOP

The combined behaviour (i.e. T-stub connection and energy dissipation device) in DBE and MCE for case A is shown in Figure 10. After combining the T-stub connection with the energy dissipation device, the connection in case A became rigid due to DBE and semi rigid due to MCE demand. On the other hand, the connection in case B became semi-rigid in both DBE and MCE demand. The stiffness of the connection due to DBE and MCE was different because the stiffness of the T-stub blind bolted connection decreases as the load increases. Residual drift limit equal to 0.1% (which indicated remedial work required) as suggested by Clifton (2010) and 0.2% (which indicated construction tolerance) in accordance to NZS 3404 (SNZ 1997) were used.

The maximum rotation and residual rotation at DBE demand for case A were 2.56 mrad and 0.09 mrad, respectively; and for case B were 4.32 mrad and 0.6 mrad, respectively. Assuming that the maximum probable rotation due to MCE was equal to 40 mrad (which is expected for very rare EQ), the residual rotation after MCE was 34.9 mrad and 32 mrad for case A and B, respectively. This would lead to a residual drift which was approximately equal to 0.009% for DBE and 3.5% for MCE in case A and 0.06% for DBE and 3.2% for MCE in case B. It was obtained that the residual drift due to DBE for both cases was less than 0.1%. Though the residual drift due to MCE for both cases was larger than 0.2%, the BRF could be easily replaced after the very rare EQ. Since the beam, column, and other connection components are elastic, the residual displacement might be reduced or at least can be brought back to the original position without replacing the main structural component. However, further investigation is needed to check the possibility of the misalignment of non-structural element and elevator shaft.
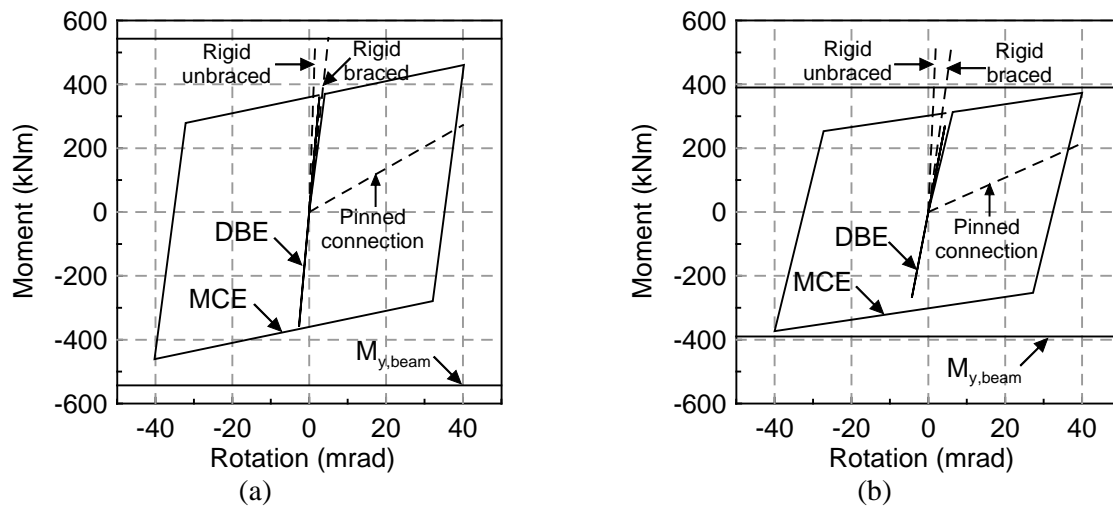

Figure 10. Moment-rotation behaviour: (a) case A; (b) case B.

6 CONCLUSIONS

The behaviour of the connections including T-stub, blind bolts, CFCHS, and replaceable BRFs as the energy dissipation was presented and a stiff semi-rigid connection could be achieved. There are several important conclusions shown as follows:

1. The dimension of the replaceable buckling restrained fuse (BRF) was chosen in a way such the beam and column will remain elastic and the maximum force is less than 60% of the ultimate capacity of the T-stub blind bolted connection.

2. It should be noted that the dimension of the headed anchor blind bolt (i.e. diameter and embedment depth) was dependent on the dimension of the steel tube.

3. FEA on cyclic behaviour of the T-stub blind bolted connection (without energy dissipation device) was presented. The result showed that negligible deterioration of strength and stiffness could be observed in both cases. Furthermore, the connection in case A remained rigid and has negligible yielding even at load levels higher than DBE. In case B, the connection was rigid in braced frame and semi-rigid in unbraced frame and has negligible yielding at load level up to DBE.

4. By including the energy dissipation device into the T-stub blind bolted connection, the drift rotation capacity was increased and the other elements such as the connections, beams, and columns were prevented from yielding. The connection in case A was rigid due to DBE and semi rigid due to MCE demand. On the other hand, the connection in case B was semi-rigid in both DBE and MCE demand. It was also obtained that the residual drift due to DBE for both cases was less than 0.1%. Though the residual drift due to MCE for both cases was larger than 0.2%, the BRF could be easily replaced after the very rare EQ because the other connection elements remained elastic.

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