



Post-tensioned self-centering moment connections with beam bottom flange energy dissipators

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ABSTRACT

This work presents results of experimental and analytical studies of self-centering moment connections. The connection subassembly consists of post-tensioned steel beams, a reinforced concrete column, and energy dissipators placed only below the beam bottom flange for simplicity of construction, ease of replacement, and no interference with the composite slab. Two types of steel energy dissipators are proposed: one includes a reduced section plate restrained by two flat plates, and the other uses cross-shaped steel plates. Cyclic tests are conducted on three full-scale post-tensioned connection subassemblies and six energy dissipators. Finite element analysis is performed to investigate the cyclic performance and likelihood of fracture at critical regions in the energy dissipators. Cyclic test results show that (1) energy dissipation, moment, and flexural stiffness of the beam in positive bending are larger than those of the beam in negative bending, (2) the location of the compression toe at the end of the beam stabilizes at the junction between the beam flange and web after an interstory drift of 1.5%, in which the gap opening angles of the beams are similar in both bending directions, and (3) the shoulder radius equal to 2.5 times plate thickness results in a premature fracture along the shape transition of the reduced section plate. This study also develops an iterative analytical procedure for predicting un-symmetrical cyclic responses of post-tensioned connection subassemblies.

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1. Introduction

Many traditional connections in steel moment resisting frames, which were fabricated following pre-Northridge construction practices, show minimal plastic deformation (e.g. 1% drift) before weld fractures at the beam-to-column interface. The poor performance observed during the 1994 Northridge Earthquake initiates a need to retrofit existing moment connections and develop new connections [1,2]. As an alternative to steel welded moment connections, post-tensioned (PT) technology has been applied to steel connections for seismic resistance [3–6]. The connections in these studies incorporate seat angles, round bars, or reduced flange plates to dissipate energy, and high-strength strands or post-tensioned bars for self-centering capability. The seat angle and reduced flange plate are bolted outside the beam flange, whereas the round bar is welded to the beam flange inner face. Energy develops by either bending the seat angle or loading axially the round bar and reduced flange plate. Thus, restraining tubes or plates are needed to eliminate buckling of the round bar

or reduced flange plate in compression. Since steel yielding is the primary energy dissipation mechanism of such devices, replacing angles or reduced flange plates damaged in seismic loadings may be difficult due to interference with the composite slab. Although round bars show good durability when resisting multiple cyclic loadings [7], the quality of field welds connecting round bars and beams is difficult to control in practice.

To eliminate the slab restraining effects, a PT connection with a discontinuous composite slab has also been investigated [7]. When a gap opens at the beam-to-column interface, the composite slab cracks freely and provides no restraint to the PT beam because the metal deck, longitudinal reinforcement, wire mesh, and concrete are discontinuous along the column centerline. For metal deck flutes perpendicular to the PT beam, the study of Collins and Filiatrault [8] also demonstrates an acceptable self-centering behavior of the PT frame.

In this regard, researchers [9] have started investigating the cyclic responses of the post-tensioned connection subassemblies with friction devices placed only below the beam bottom flange to eliminate interference with the composite slab. A bottom flange friction device consists of a vertically oriented slotted plate that is shop welded to the beam bottom flange and outer angles that are field bolted to the column. The friction plate material between the angle and slotted plate is ASTM B-19 UNS brass. The test

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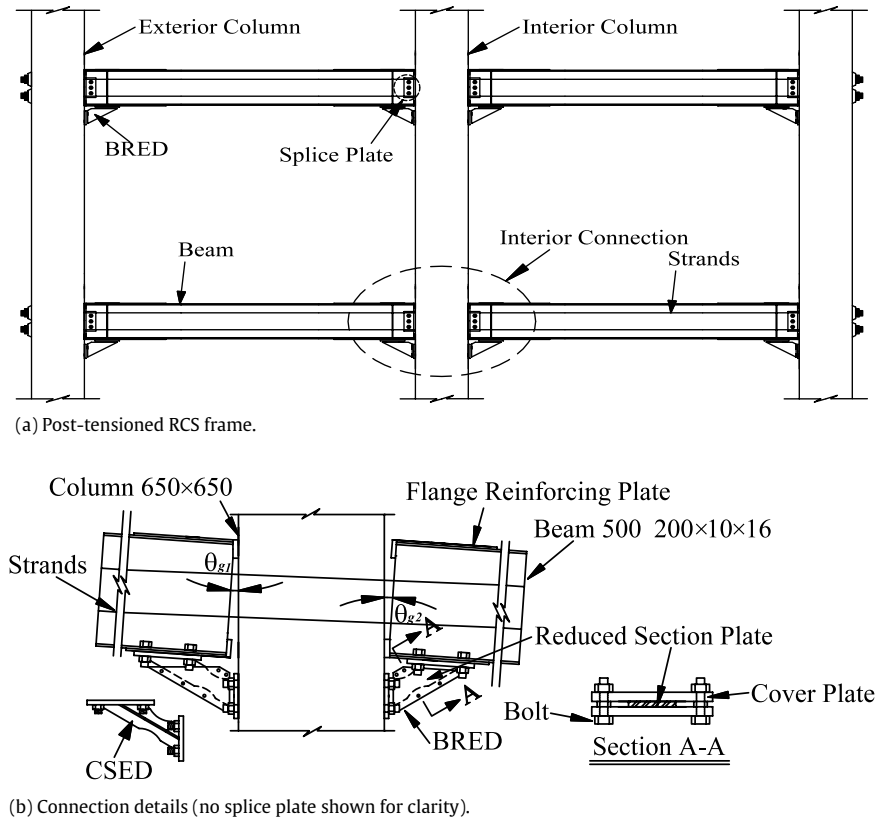


Fig. 1. Proposed post-tensioned frame with bottom flange energy dissipators.

results demonstrate that the bottom flange friction device in the PT connection provides good energy dissipation with no stiffness after decompression. The study [10] adopts a similar concept but uses metallic yielding devices rather than friction devices. In this case, both energy dissipation and post-yield stiffness of the device can be developed in the PT connection; piercing noise caused by bolt slippage during friction [11] can also be eliminated. Two types of steel energy dissipators are proposed: one is composed of a reduced section plate restrained by two flat plates—a configuration similar to that proposed by Inoue et al. [12]; and the other is made of cross-shaped steel plates. The energy dissipator is field bolted to the column and beam after beams are post-tensioned to the column. It is easier to install and replace the bottom flange energy dissipator than the restrained steel plate [6,13] due to no welded joint between the dissipator and connection. Moreover, when a reinforced concrete column instead of a steel column is specified, the use of a bottom flange energy dissipator reduces connection constructability and cost.

The work presented in this paper is part of a research program on the cyclic behavior and column restraining effects of a post-tensioned self-centering moment frame, which is composed of PT concrete columns and PT steel beams [14]. Therefore, concrete columns are selected instead of steel columns in this study. Three post-tensioned connection subassemblies with proposed bottom flange energy dissipators are tested to investigate cyclic behavior. Since the behavior is expected to differ from that of a post-tensioned connection with energy dissipators placed on the top and bottom beam flanges, this study develops an iterative analytical procedure for predicting the un-symmetrical cyclic behavior. Furthermore, six uni-axial tests and finite element analyses of energy dissipators are performed to evaluate energy dissipation capability and likelihood of fracture associated with various dimensions of the reduced section plate.

2. Post-tensioned connection subassembly behavior

Fig. 1(a) presents the geometric configuration of a frame incorporating the proposed bottom flange energy dissipators. The steel beam web is first positioned to a splice plate, which is embedded in the concrete column and has slotted holes for bolted connection. Beams are post-tensioned to columns via high-strength steel strands before the energy dissipators are bolted below the beam bottom flange and column. The experimental program involved testing three full-scale subassemblies, each of which was composed of a reinforced concrete column (650×650 mm), two steel beams ($H500 \times 200 \times 10 \times 16$ mm), and bottom flange energy dissipators (Fig. 1(b)). Specimens 1 and 2 used buckling-restrained energy dissipators (BREDs), which have a reduced section plate restrained by two cover plates; Specimen 3 used cross-shaped energy dissipators (CSRDs), which have a reduced section plate welded with two plates in the transverse direction.

Fig. 2 presents the moment versus gap opening angle relationship of the proposed post-tensioned connection. Notation θ_{g1} represents the angle of the gap opening at the beam-to-column interface. The total beam moment [Fig. 2(c)] is contributed from the strands [Fig. 2(a)] and an energy dissipator [Fig. 2(b)]. Since no energy dissipator is located on the beam top flange, the hysteretic loop is un-symmetrical and beam decompression in the negative bending (point 1') occurs earlier than that in the positive bending (point 1). The beam moment at a decompression point is called the decompression moment. When the beam is in positive bending (moving upward), the energy dissipator is under tension. Once the decompression moment is exceeded at point 1, the response follows line 23 and the energy dissipator yields at point 2. When the load is reversed at point 3, the energy dissipator yields after point 4 is reached and the gap closes at point 5, generating a self-centering response. Similar responses occur with small energy dissipation when the beam is in negative bending.

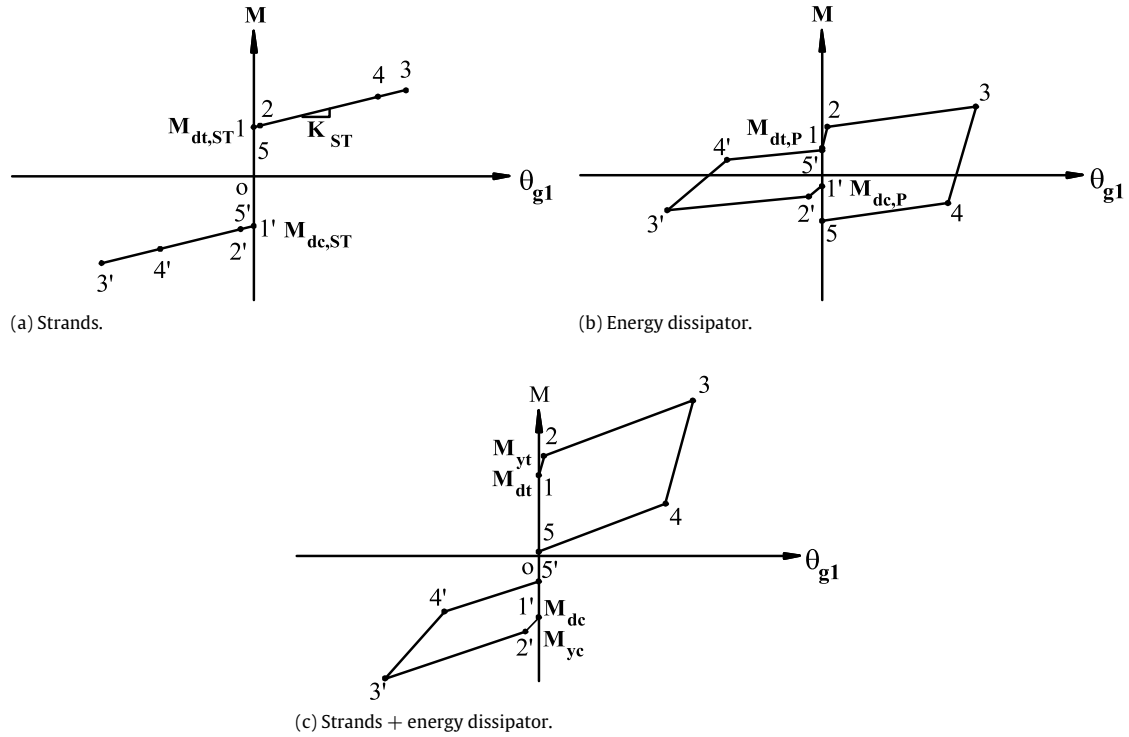


Fig. 2. Moment provided by strands and an energy dissipator.

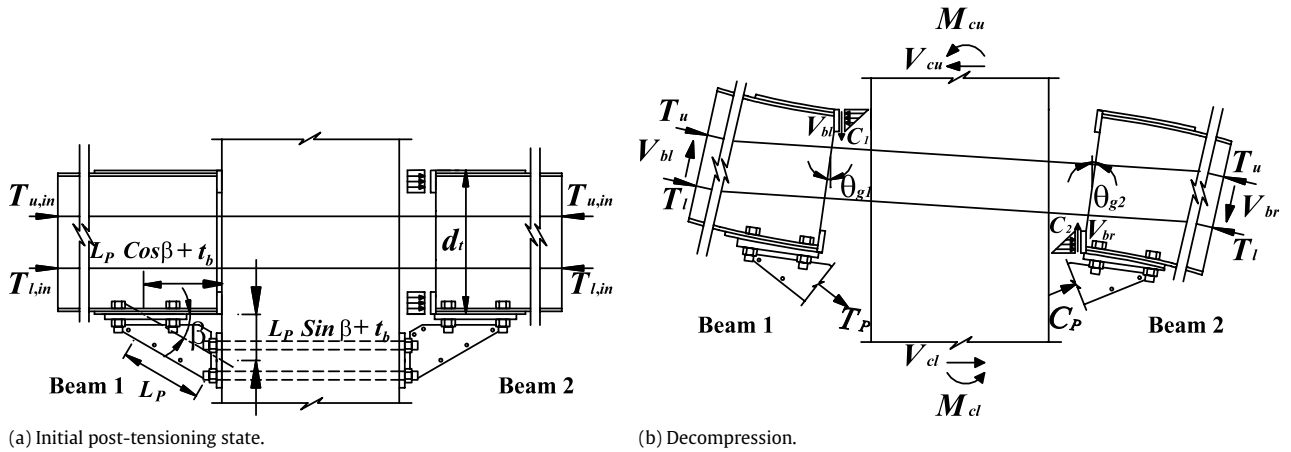


Fig. 3. Free-body diagram.

2.1. Decompression moment

Fig. 3(a) shows the free body diagram of the connection subassembly in the initial post-tensioning state. The PT connection behaves as a fully restrained moment connection provided the beam moment is less than the decompression moment at the beam-to-column interface. The decompression moment, M_{dc} , of the beam during negative bending is calculated as:

$$M_{dc} = M_{dc,ST} + M_{dc,P} \\ = \left[(T_{u,in} + T_{l,in}) \left(\frac{d_b}{2} - t_f \right) \right] \\ + [C_p \cos \beta (L_p \sin \beta + t_b + t_r + t_f)] \quad (1)$$

where d_b is the beam depth; t_f is the beam flange thickness; t_b is the base plate thickness of the energy dissipator; t_r is the flange reinforcing plate thickness; L_p is the length of the reduced section plate; β is the angle between the beam flange and energy

dissipator; $T_{u,in}$ and $T_{l,in}$ are the initial forces in the upper and lower strands, respectively. Compressive force, C_p , in the energy dissipator can be estimated based on its axial deformation, Δ_{in} , which is computed from initial shortening of the beam section where the energy dissipator is connected:

$$\Delta_{in} = \int_0^{L_p \cos \beta + t_b - t_s} \frac{T_{in}}{E_s A(x)} \cos \beta dx \quad (2)$$

where T_{in} is the total initial post-tensioning force ($=T_{u,in} + T_{l,in}$) in the beam; t_s is the thickness of the bearing plate at the beam end, and $A(x)$ is the cross-sectional area along the beam length. The yield force and ultimate force of the reduced section plate are determined as the smallest sectional area times yield strength and ultimate strength, respectively. The corresponding axial yield deformation and ultimate deformation are calculated by integrating the strain over the entire length of the reduced section plate L_p . The axial force–deformation relationship is constructed using a bi-linear relationship.

Under continued loading, decompression of the beam in positive bending occurs with a decompression moment:

$$\begin{aligned} M_{dt} &= M_{dt,ST} + M_{dt,P} \\ &= \left[(T_{u,in} + T_{l,in}) \left(\frac{d_b}{2} - t_f \right) \right] \\ &\quad + [T_p \cos \beta (L_p \sin \beta + t_b + d_t - t_r - t_f)] \end{aligned} \quad (3)$$

where d_t is the beam depth plus flange reinforcing plate thickness, and T_p is the tensile force in the energy dissipator, that can be computed based on its axial deformation, Δ_{in} . After decompression, the forces in the strands and energy dissipators can be estimated based on the following procedure, which can also be used for estimating the moment–drift relationship of the connection subassembly with different gap opening angles at beam ends [Fig. 3(b)].

2.2. Iterative analytical procedure

Based on the previous studies [3,5,7,11,13], the compression toe at the beam end is assumed at the junction between the beam flange and web. The following procedure calculates the beam moment of the PT connection subassembly for a specified interstory drift θ in two beams—one subjected to positive bending and the other subjected to negative bending (Fig. 3(b)):

1. Assume gap opening angles, θ_{g1} and θ_{g2} , in Beam 1 and Beam 2, respectively.
2. Compute strand force T_{ST} based on strand elongation:

$$\begin{aligned} T_{ST} &= T_u + T_l = T_{in} + N_{ST} \Delta T \\ &= T_{in} + N_{ST} \left[\frac{(d_b/2 - t_f) (\theta_{g1} + \theta_{g2})}{L_{ST}} \right. \\ &\quad \times \left. \left(1 - \frac{N_{ST} A_{ST}}{A_b + N_{ST} A_{ST}} \right) \right] E_{ST} A_{ST} \end{aligned} \quad (4)$$

where A_{ST} is the sectional area of a tendon, that contains four strands; N_{ST} (=4) is the number of tendons; E_{ST} is the elastic modulus of the strands, and L_{ST} is the length of the strands.

3. Compute tensile deformation, Δ_t , and compressive deformation, Δ_c , of the energy dissipators:

$$\Delta_t = \Delta_{in} + (L_p \sin \beta + t_b + d_t - t_r - t_f) \theta_{g1} \cos \beta \quad (5)$$

$$\Delta_c = \Delta_{in} + (L_p \sin \beta + t_b + t_r + t_f) \theta_{g2} \cos \beta. \quad (6)$$

Deformation in Eqs. (5) and (6) includes two parts. The first part, Δ_{in} , is the dissipator deformation from Eq. (2). The second part is the dissipator deformation based on rigid body rotation of the beam about the compression toe. The term in the parenthesis is the distance between the position of the compression toe in the beam and the dissipator-to-column joint [Fig. 3(a)]. Axial forces, T_p and C_p , in the energy dissipators can be determined based on the axial force–deformation relationship described earlier.

4. Compute moment M_1 of Beam 1 and M_2 of Beam 2 associated with the respective gap opening angles:

$$\begin{aligned} M_1 &= \left[T_{ST} \left(\frac{d_b}{2} - t_f \right) \right] + T_p \cos \beta (L_p \sin \beta + t_b + d_t - t_r - t_f) \\ &= M_{t,ST} + M_{t,P} \end{aligned} \quad (7)$$

$$\begin{aligned} M_2 &= \left[T_{ST} \left(\frac{d_b}{2} - t_f \right) \right] + C_p \cos \beta (L_p \sin \beta + t_b + t_r + t_f) \\ &= M_{c,ST} + M_{c,P}. \end{aligned} \quad (8)$$

5. Calculate the interstory drift $\bar{\theta}_1$ of Beam 1 and $\bar{\theta}_2$ of Beam 2 using the following equations:

$$\bar{\theta}_1 = \frac{M_1}{K_{TE}} + \theta_{g1} \quad (9)$$

$$\bar{\theta}_2 = \frac{M_2}{K_{TE}} + \theta_{g2} \quad (10)$$

where K_{TE} is the elastic flexural stiffness of the connection subassembly described in the following section. Check whether computed interstory drifts $\bar{\theta}_1$ and $\bar{\theta}_2$ are equal to the specified interstory drift θ ; otherwise, iterate over new θ_{g1} and θ_{g2} by returning to step 1.

2.3. Flexural stiffness of the post-tensioned connection subassembly

The precompression provided by the strands ensures full contact between the beam and column before decompression. The moment–drift relationship exhibits elastic behavior, and the initial flexural stiffness is similar to that in a fully restrained moment connection [3]. Hence, the elastic flexural stiffness of the post-tensioned beam, K_b , is approximated using that for a fully restrained beam [15]. The elastic flexural stiffness provided by the energy dissipator is estimated based on the ratio of moment provided by the energy dissipator to the post-tensioned beam at decompression [6]:

$$K_{EP} = K_b \frac{M_{dc,P}}{M_{dc,ST}} \quad (11)$$

where $M_{dc,P}$ and $M_{dc,ST}$ are from Eq. (1). Since the column and panel zone remain elastic throughout the test, the elastic flexural stiffness of the connection subassembly, K_{TE} , is calculated as:

$$K_{TE} = \frac{1}{\frac{1}{K_c} + \frac{1}{K_{PZ}} + \frac{1}{K_b + K_{EP}}} \quad (12)$$

where K_c and K_{PZ} are the elastic flexural stiffness of the column and panel zone, respectively, and can be found elsewhere [16].

2.4. Design criteria for energy dissipator

An energy dissipator of the buckling-restrained type is adopted to prevent Euler buckling. The energy dissipator developed in this study is more compact than that devised previously [12]. As shown in Fig. 4 for Specimens 1 and 2, the core is made of a steel reduced section plate coated by a friction-reduced material and encased in a restraining sheath made of two steel cover plates. The sheath can also be built from a double steel T member whose design and performance have been verified by a recent experimental investigation [6,12,13]. Each end of the core is welded to a flat plate which, in turn, is connected to the beam flange or column by ASTM A490 bolts. The center-reduced section is aimed to confine yielding area in the core. As in usual practice [12], the dissipator inclination is 30° relative to the horizon. The core is sized based on an expected moment M_p ($= \alpha M_{np}$, where $\alpha \approx 0.2$ –0.3) and tensile strain, ε_p , at a specified interstory drift (e.g. 3%). The strain, ε_p , is computed according to the axial deformation (Δ_t) and length (L_p) of the energy dissipator (Eq. (5)). The design criteria are that at this interstory drift yielding or fracture concentrates only in the energy dissipator while the beams, columns, and strands remain elastically. The moment, M_p , must always be smaller than decompression moment generated by the initial PT force to ensure full re-centering on load reversal. The steps required to determine the shape of the reduced section plate are similar to those listed in the previous study [6] except for calculation of the sectional area. For the proposed energy dissipator, the narrowest sectional area, A_p , in the core is calculated based on:

$$A_p = \frac{\alpha M_{np}}{\sigma_p \cos \beta (L_p \sin \beta + t_b + d_t - t_r - t_f)} \quad (13)$$

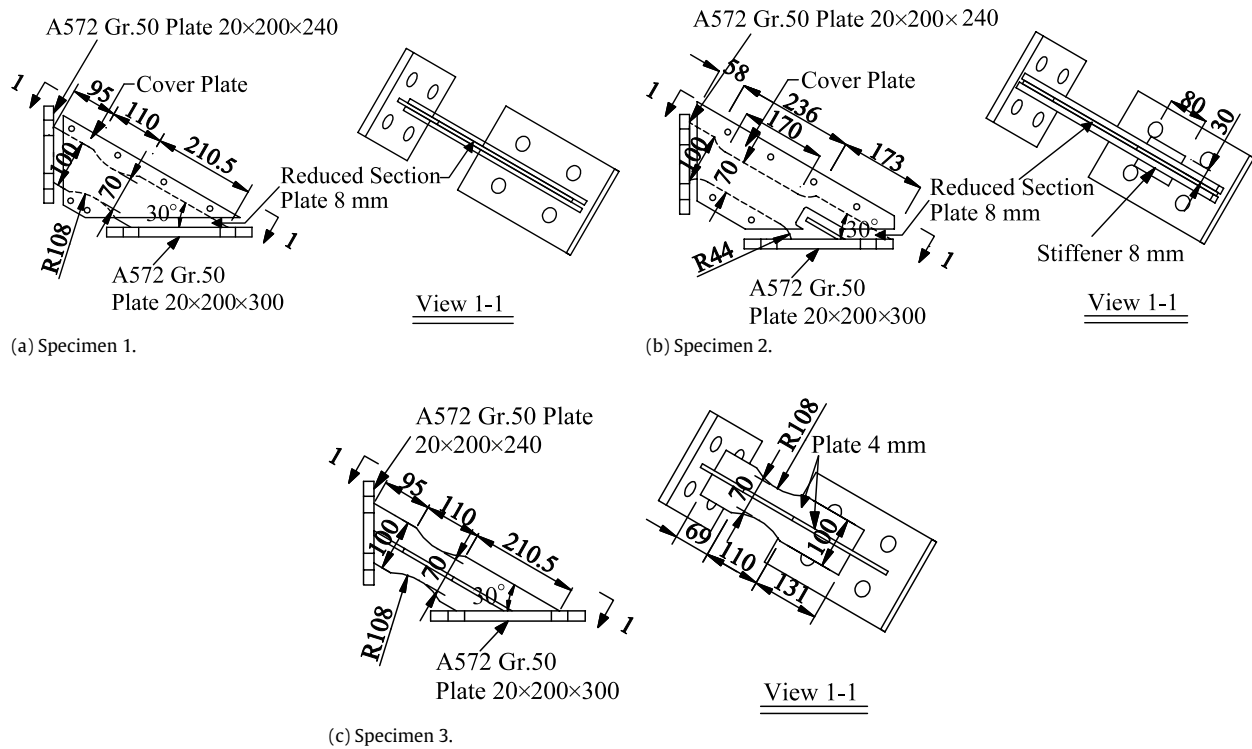


Fig. 4. Energy dissipator details.

Table 1
Specimen behavior prediction.

(a) Moment and strain

Specimen no.	T_{in} (kN)	$\frac{T_{in}}{T_u}$ ^a	K_{TE} (kN-m)	$\frac{M_{dt}}{M_{np}}$ or $\frac{M_{dc}}{M_{np}}$		$\frac{M_{3\%}}{M_{np}}$		ϵ_P (%)
				Pos.	Neg.	Pos.	Neg.	
1	800	0.27	85 868	0.32	0.30	0.70	0.49	22.3
2	1060	0.36	85 868	0.47	0.41	0.78	0.60	9.3
3	800	0.27	91 764	0.32	0.30	0.70	0.49	22.3

(b) Gap opening angles

Specimen no.	1% Drift		1.5% Drift		2% Drift		3% Drift	
	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
1	0.53 ^c	0.66	0.99	1.13	1.45	1.59	2.37	2.54
2	0.45	0.57	0.85	0.92	1.37	1.50	2.29	2.44
3	0.53	0.66	0.99	1.14	1.45	1.59	2.37	2.54

^a $T_u = 2937$ kN.^b $M_{np} = 750$ kN-m.^c Unit for gap opening angle is $\times 0.01$ rad.

where σ_P is the stress corresponding to the strain ϵ_P and M_{np} is the nominal moment capacity of the beam. Instead of using a restrained steel plate for energy dissipation, a cross-shaped steel plate device is used in Specimen 3. In each specimen, the same cross-sectional area, A_p , and length, L_p , are used for all energy dissipators so that failure of each dissipator can be investigated during the connection subassembly test.

3. Connection subassembly test and analysis

The experimental program consisted of tests of three full-scale subassemblies, which had two steel beams post-tensioned to a reinforced concrete column. Specimens 1 and 2 had a BRED placed below the beam bottom flange. Specimen 1 adopted a circular-cut section in the ASTM A36 steel plate [Fig. 4(a)], and Specimen 2 used a coupon-shaped SS 400 plate [Fig. 4(b)]. Two ASTM A572 Gr. 50 cover plates were connected using bolts and were used to prevent

buckling of the reduced section plate. Specimen 3 utilized A36 steel plates with a minimum sectional area in the middle length of the device [Fig. 4(c)]. All specimens had four tendons running parallel to the beam web; each tendon contained four 13 mm diameter seven wire, uncoated, low-relaxation ASTM A416 Grade 270 strands. The modulus of elasticity and tensile strength according to the manufacturer were 195 GPa and 1860 MPa, respectively. The stiffener of the beam at locations where the energy dissipator was connected was utilized to eliminate beam web crippling according to Chapter K of AISC-LRFD specifications [17]. Four fully-tensioned A490 bolts (28 mm in diameter) were required to prevent slippage between the beam and energy dissipator before reaching the ultimate capacity. Table 1(a) lists total initial post-tensioning force, T_{in} , elastic flexural stiffness of the post-tensioned connection subassembly, K_{TE} , decompression moment, M_{dt} and M_{dc} , moment at an interstory drift of 3%, $M_{3\%}$, and corresponding maximum tensile strain, ϵ_P , in the reduced section plate. High axial strain in the reduced section plate was intentionally designed to

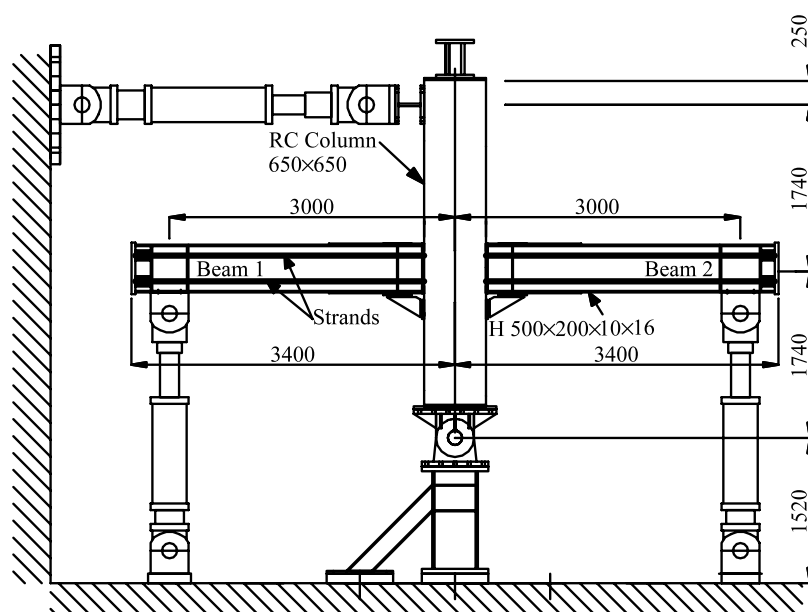


Fig. 5. Test setup.

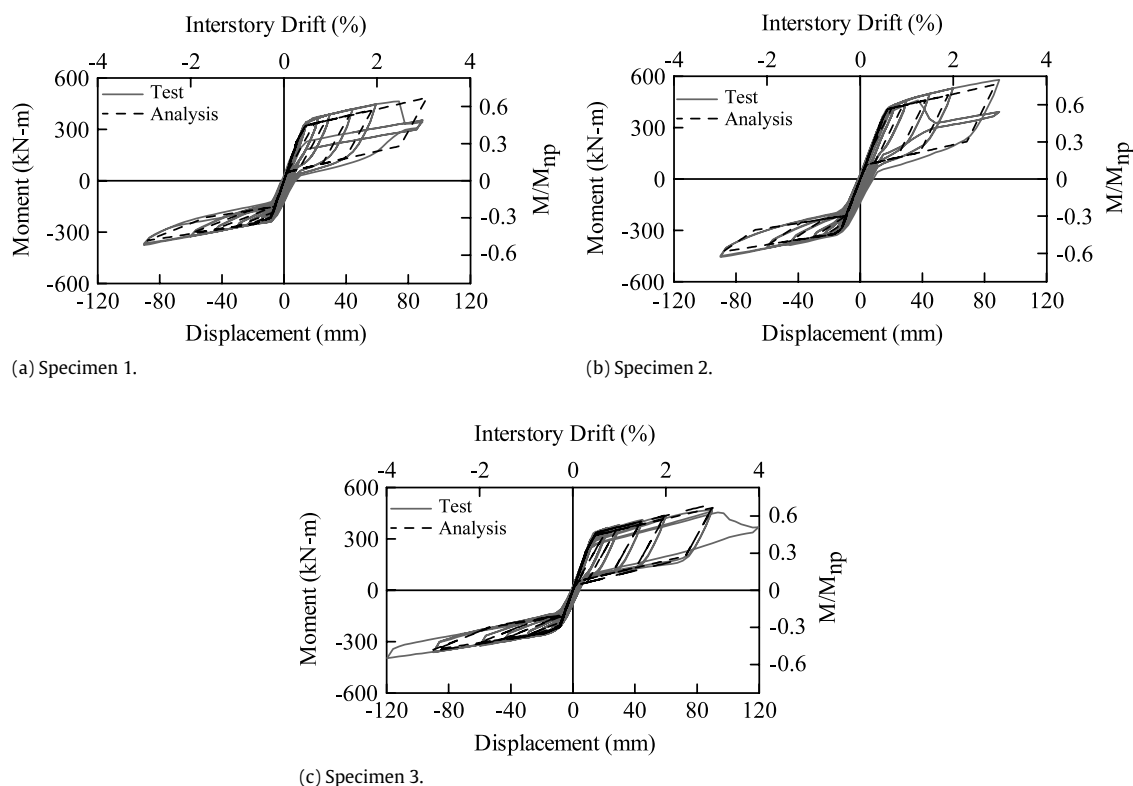


Fig. 6. Beam moment versus drift relationship.

discover the potential failure of energy dissipators during tests. Table 1(b) lists the gap opening angles of the beams predicted based on the proposed iterative analysis; the angles differ by 10% after an interstory drift of 1.5%.

Each specimen was tested in the setup (Fig. 5) by displacing actuators at both ends of beams through a series of displacement cycles, consistent with the AISC loading protocol [18]. Fig. 6 shows the relationships between beam tip deflection and moment for the three specimens. The BRED of Specimen 1 fractured toward the first cycle of an interstory drift of 3%. The beam moment, which was then provided by only strands, contributed to approximately

half of the desired moment. The cover plate was removed after completing the second cycle of an interstory drift of 3% to locate fracture. Fig. 7(a) shows a crack in the narrowest section of the reduced section plate. The BRED of Specimen 2 fractured at the weld toe between the reduced section plate and base plate [Fig. 7(b)] when the beam moved during the second cycle of an interstory drift of 3%. The CSED of Specimen 3 yielded following decompression and buckled in compression [Fig. 7(c)] at an interstory drift of 1.5%. Buckling amplitude increased with drift, and the fracture occurred in the narrowest section of the plate during the first cycle of an interstory drift of 4% [Fig. 7(d)].

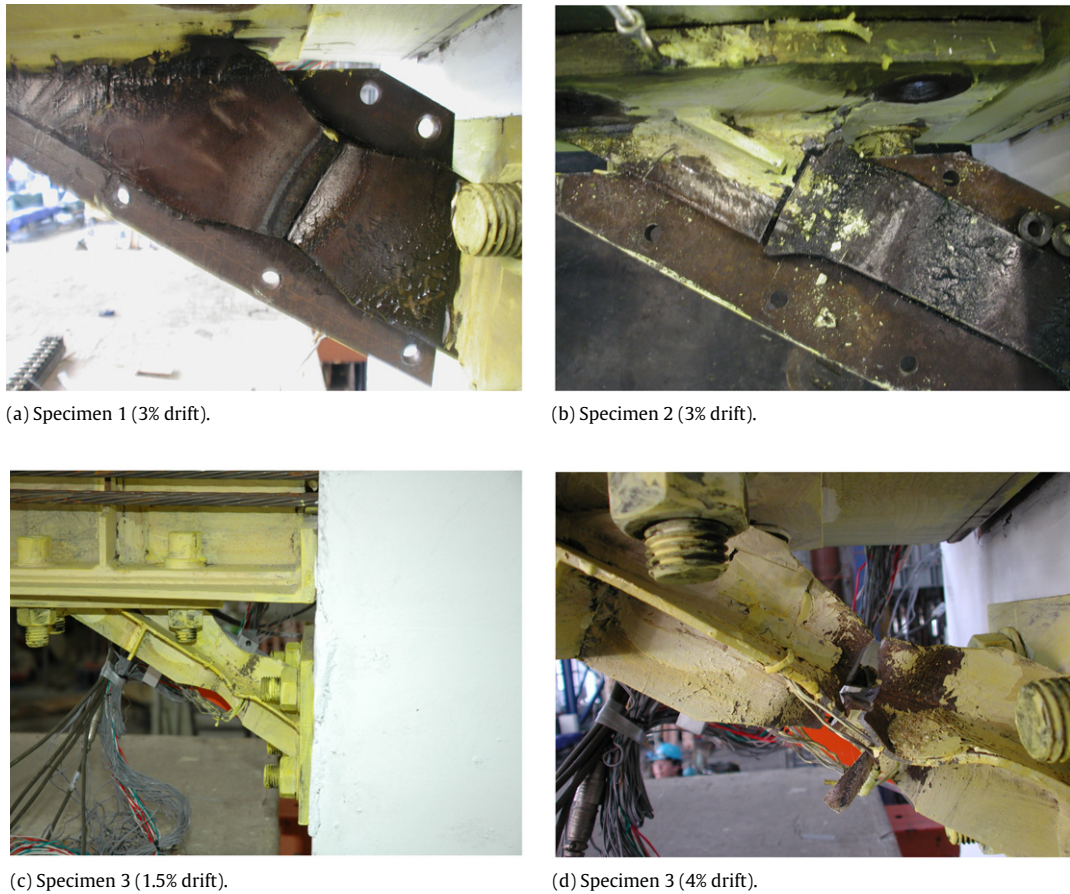


Fig. 7. Specimen failure modes.

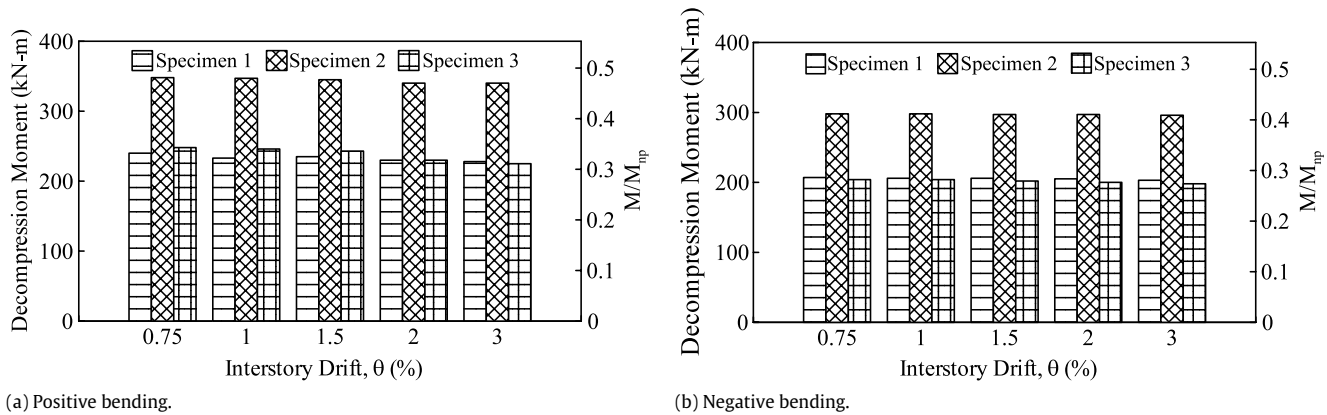


Fig. 8. Decompression moment versus interstory drift relationship.

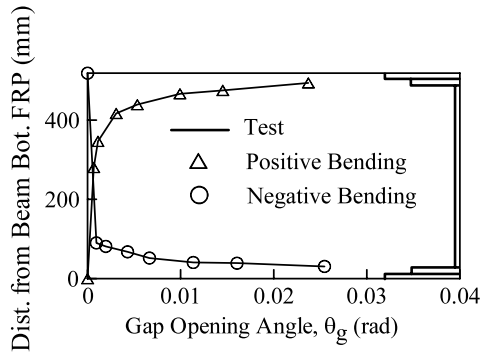
Table 2
Test response.

Specimen no.	T_{in} (kN)	$\frac{T_{in}}{T_u}$	K_{TE} (kN-m)	$\frac{M_{dt}}{M_{np}}$ ^a or $\frac{M_{dc}}{M_{np}}$		$\frac{M_{sg}}{M_{np}}$	
				Pos.	Neg.	Pos.	Neg.
1	800	0.27	76 437	0.34	0.31	–	0.49
2	1060	0.36	75 485	0.48	0.42	0.78	0.60
3	800	0.27	77 115	0.33	0.31	0.68	0.49

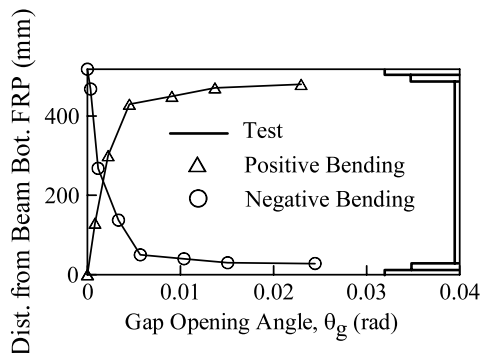
^a M_{np} = Nominal plastic moment of the beam.

Fig. 8 shows the decompression moment for all specimens bending in both directions. The decompression moment was computed as the actuator force times the distance to the column face when the strand force increased from an initial value in

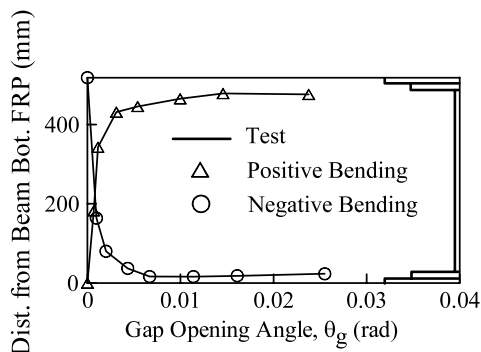
each drift cycle. As predicted (Table 1(a) versus Table 2), the decompression moment of the beam in positive bending is always higher than that in negative bending. Specimen 2 has the highest value for decompression moment due to the largest initial post-



(a) Specimen 1



(b) Specimen 2



(c) Specimen 3

Note: FRP stands for flange reinforcing plate placed outside beam flange

Fig. 9. Location of compression toe.

tensioning force in the beam (Table 1(a)). Fig. 9 shows variation of the compression toe at the beam end. As the assumption made in the iterative analytical procedure, the compression toe location stabilizes at the junction between beam flange and web at a gap opening angle of about 0.01 rad, corresponding to an interstory drift of 1.5%. The measured gap opening angle of the beam in negative bending (θ_g^-) is always higher than that in positive bending (θ_g^+); however, these two angles are close after an interstory drift of 1.5%, leading to a ratio (θ_g^+/θ_g^-) of 0.9–1 (Fig. 10). This fact indicates that iterative analysis can be simplified by assuming the same gap opening angles of beams subjected to positive and negative bending, respectively. The simplified analysis reasonably predicts the hysteretic responses of the PT connection subassemblies (Fig. 6). Note that buckling of the CSEDs occurred at an interstory drift of 1.5%; consequently, the axial force–deformation relationship of the device was obtained from finite element analysis rather than through calculation.

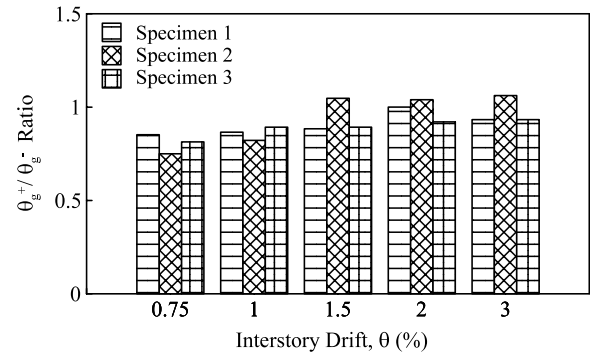


Fig. 10. Gap opening angle ratio versus interstory drift relationship.

4. BRED test and analysis

The test results of Specimens 1 and 2 showed that strain concentration in the narrowest section of the reduced section plate leads to BRED fracture and a coupon-shaped plate can exclude strain concentration in the section for energy dissipation except for the shape transition. Hence, experimental and analytical studies on this type of the energy dissipator were further conducted. Six BREDS, each composed of an 8-mm thick reduced section plate and two cover plates (Fig. 11), were tested axially to investigate their cyclic behavior. The parameters were length of the reduced section, L_1 , shoulder radius, R , and material type (Table 3). Steel SS 400 had an yield strength similar to ASTM A36 steel and much less ultimate strain, which is the strain at the onset of necking. Except for BRED-1, which was subjected to monotonic loading, remaining five BREDS were subjected to cyclic loading. The displacement cycles were consistent with deformation of a BRED measured during the connection subassembly test. The cyclic responses of the BREDS (Fig. 12) indicate that the energy dissipation was stable until the reduced section plate fractured. The reduced section plate of BRED-2 buckled outside the region restrained by the cover plates when the load was reversed at a displacement cycle of 22 mm [Fig. 12(a)]. The peak force of BRED-2 at each displacement cycle was close to that of BRED-1 obtained during the monotonic loading test. Both devices fractured close to Section A-A (Table 3). For BRED-1, BRED-2, and BRED-5, which have R values no smaller than 4 times plate thickness ($t_p = 8$ mm), fractures occurred close to Section A-A (Table 3). However, for BRED-3 and BRED-4, which have R values equal to $2.5t_p$, fractures occurred in Section B-B (Table 3). The device BRED-6 was tested symmetrically twice with maximum deformation of 7 mm, resulting in maximum tensile strain of 4% without fracture; compression force was 9% higher than tensile force at the same displacement level [Fig. 12(d)].

Finite element analysis was used to predict cyclic response of a BRED, estimate maximum tensile strain in the reduced section plate, and determine the likelihood of fracture associated with different shoulder radius. The reduced section plate was modeled using four-node shell elements, S4R, in the computer program ABAQUS [19], and allowed to move axially. Fig. 13 shows the tensile strains of the BREDS obtained for different displacement levels. Each displacement level has four bars. The first bar represents average strain in Section A-A obtained using the finite element model. The second bar represents the strain computed by integrating the strain over the entire length to reach a required displacement. The third bar represents the strain obtained from displacement divided by length L_1 (Table 3). The fourth bar represents the strain obtained from displacement divided by length L_2 (Table 3). Using deformation divided by length L_1 is an easy approach and reasonably predicts the plate strain in the finite element model.

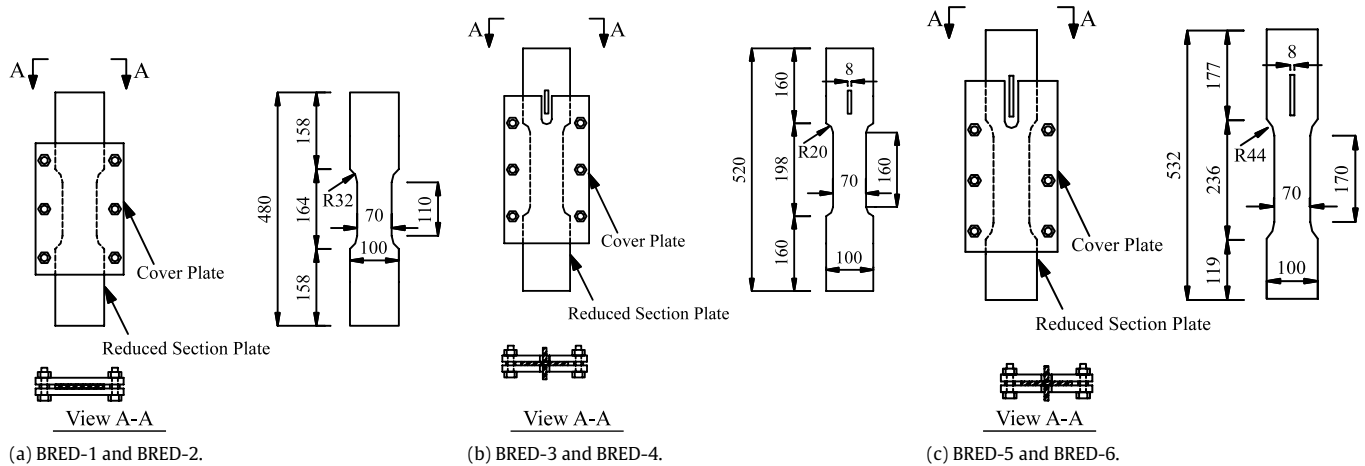


Fig. 11. BRED details.

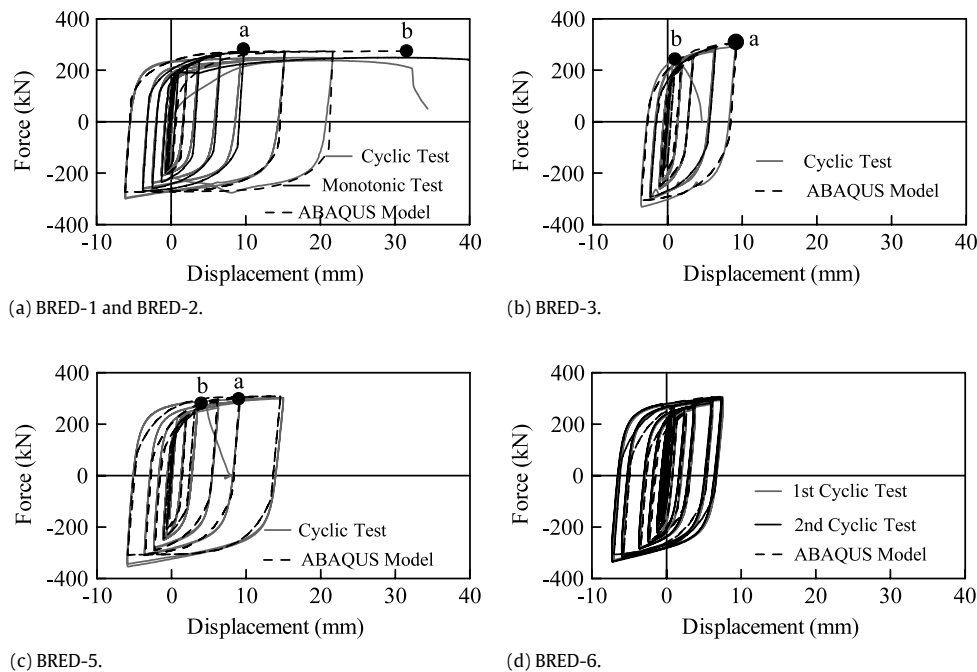


Fig. 12. BRED uni-axial cyclic test result versus finite element analysis.

Table 3
BRED design parameter.

BRED no.	Material	Yield strength (MPa)	Ultimate strain (%)	L_1 (mm)	L_2 (mm)	R (mm)	Fracture section	Loading	Notation
BRED-1	A36	320	25	110	164	32	A-A	monotonic	
BRED-2	A36	320	25	110	164	32	A-A	cyclic	
BRED-3	SS 400	370	15	160	198	20	B-B	cyclic	
BRED-4	SS 400	370	15	160	198	20	B-B	cyclic	
BRED-5	SS 400	370	13	170	236	44	A-A	cyclic	
BRED-6	SS 400	370	13	170	236	44	–	cyclic, twice	

The rupture index (RI) is equal to the product of a material constant and the equivalent plastic strain divided by the strain at a ductile fracture, which is given by Hancock and Mackenzie [20]. Fracture initiation is caused by high tensile triaxial stress, which results in accumulated damage. Locations with high RI values have a greater potential for fracture. Because the shell element does not have through-thickness stresses, the rupture index (RI)

obtained from the analysis approximates stress concentration in the reduced section plate. Fig. 14 shows RI distributions in Section A-A and Section B-B at a tensile displacement of 9.75 mm (point a in Fig. 12) and before fracture (point b in Fig. 12). For BRED-2, which uses $4t_p$ as the shoulder radius, the maximum RI values in Section A-A and Section B-B are similar at point a [Fig. 14(a)]. However, the maximum RI value in Section A-A is much higher

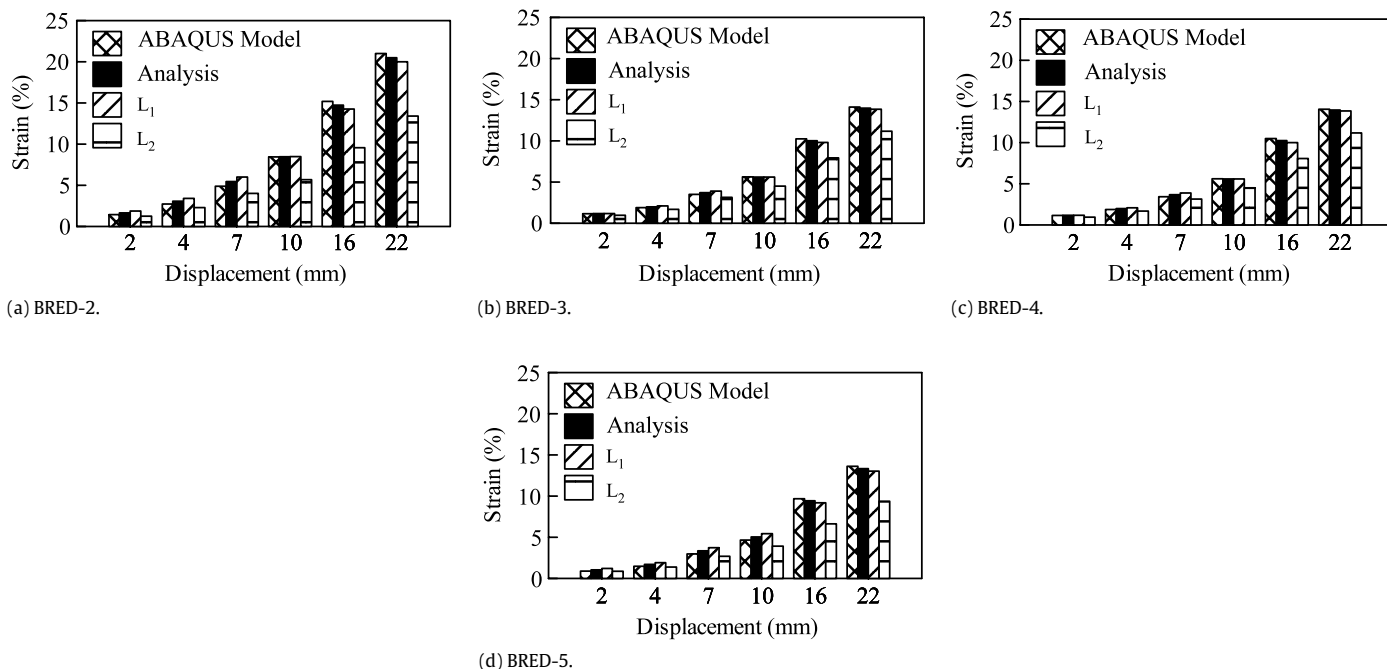


Fig. 13. BRED tensile strain.

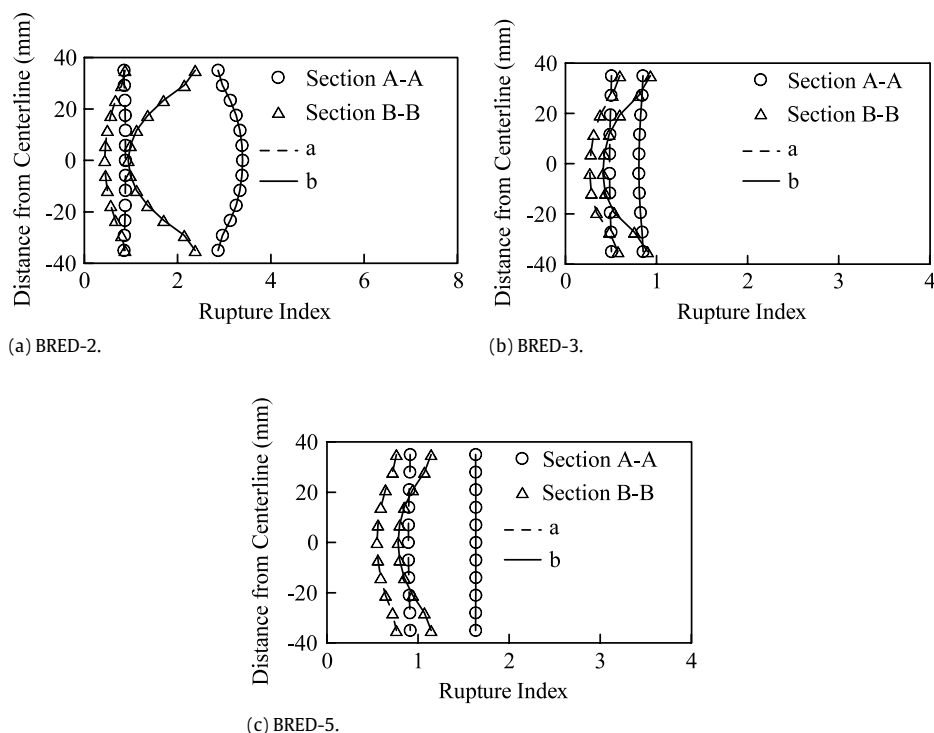


Fig. 14. BRED rupture index distributions.

than that in Section B-B at point b, indicating that Section A-A is the location for potential fracture. For BRED-3, which has a shoulder radius of $2.5t_p$, the maximum RI value is higher at the edge of Section B-B than that for Section A-A for both displacement levels [Fig. 14(b)], indicating that both edges in Section B-B are the locations for potential crack initiation. The maximum tensile strain before fracture (point a in Fig. 12(b)) is about 5%, which is 33% of the ultimate strain (Table 3). The distributions of RI [Fig. 14(c)] for BRED-5, which has a shoulder radius of $5.5t_p$, clearly show that

Section A-A is the location for potential fracture, corresponding to the location of fracture observed in the test. The maximum tensile strain at a displacement cycle of 16 mm [Fig. 12(c)] is about 10%, which is 77% of the ultimate strain (Table 3).

5. Conclusions

Three post-tensioned connection subassemblies, which utilized high-strength strands to provide self-centering capability and bottom flange energy dissipators to provide energy dissipation, were

investigated experimentally and analytically. The moment–drift relationship of the connection subassembly had a higher moment and energy dissipation for a beam under positive bending than under negative bending. Although the cyclic response was unsymmetrical, the location of the compression toe at the end of the beam stabilized at the junction between the beam flange and web after an interstory drift of 1.5%, in which the gap opening angles of the beams approached a similar value in both bending directions. Assuming a fixed location of the compression toe following decompression and the same gap opening angles of beams in both positive and negative bending simplified the iterative analytical steps and reasonably predicted test responses.

Six BREDs, each composed of a reduced section plate and two cover plates, were tested and analyzed to evaluate their cyclic performance. Two cover plates connected by bolts were effective in preventing buckling of the reduced section plate under compression. When the shoulder radius was 2.5 times plate thickness, test results and distributions of the rupture index from finite element analyses demonstrated that a fracture occurred along the shape transition at the strain level, about 33% of the ultimate strain. As long as the shoulder radius was no smaller than four times plate thickness, the fracture along the shape transition could be eliminated and the strain at fracture could reach 77% of the ultimate strain.

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