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## Performance of a damage-protected beam–column subassembly utilizing external HF2V energy dissipation devices

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### SUMMARY

Ductile-jointed connections, which generally require some form of supplementary energy dissipation to alleviate displacement response, typically employ mild steel energy dissipation devices. These devices run the risk of low-cycle fatigue, are effective only for peak cycles that exceed prior displacements, are prone to buckling, and may require replacement following an earthquake. This study presents an experimental investigation employing an alternative to mild steel: a high force-to-volume (HF2V) class of damper-based energy dissipation devices. Tests are performed on a near full-scale beam–column joint subassembly utilizing externally mounted compact HF2V devices. Two configurations are considered: an exterior joint with two seismic beams and one gravity beam framing into a central column, and a corner joint with only one seismic beam and one gravity beam framing into a column. Quasi-static tests are performed to column drifts up to 4%. The experiments validate the efficacy of the HF2V device concept, demonstrating good hysteretic energy dissipation, and minimal residual device force, allowing ready re-centring of the joint. The devices dissipate energy consistently on every cycle without the deterioration observed in the yielding steel bar type of devices. The effectiveness of the HF2V devices on structural hysteretic behavior is noted to be sensitive to the relative stiffness of the anchoring elements, indicating that better efficiency would be obtained in an embedded design. Copyright © 2008 John Wiley & Sons, Ltd.

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## INTRODUCTION

Research and development of precast concrete jointed and rocking structures has gained considerable momentum over the past two decades, with significant research on the so-called PRESSS systems [1], damage-avoidance design (DAD) [2–4] in the United States and New Zealand [5–7]. These systems, designed to accommodate inelastic behaviour by rocking at specially detailed joints, have proven to provide a level of seismic resistance comparable to current standards while remaining almost (in the case of PRESSS) or essentially (in the case of DAD) damage-free. Furthermore, such systems do not suffer excessive residual displacement, a common occurrence in conventional sacrificially designed systems, which often leads to complete loss of the structure.

Precast jointed and rocking systems exhibit non-linear response by connection opening instead of the conventional sacrificial, energy dissipating, formation of a plastic hinge. As a result, they have markedly less inherent energy dissipation than ductile monolithic systems [8, 9]. Therefore, it is desirable to provide supplemental energy dissipation devices to these connections to reduce displacement response from earthquakes.

An early application presented by Stanton *et al.* [10] utilized mild steel rods running across the connection and grouted in ducts, which were designed to yield in tension and compression. Although test results revealed satisfactory hysteretic energy dissipation, because the bond between the grout and steel deteriorated, some stiffness and strength degradation was observed. In addition, repeated cycles led to bar fracture in some tests due to low-cycle fatigue, showing an inherent weakness in any sacrificial yielding-steel energy dissipation approach, be it a device or by sacrificial design.

Subsequent research has highlighted alternative mild steel energy dissipation devices, bolted externally across the joint region. Bradley *et al.* [5] have demonstrated that externally mounted steel yielding devices can provide a stable hysteric performance with negligible stiffness or strength degradation. However, due to low-cycle fatigue and residual stresses these devices would still have to be replaced following an earthquake. In addition, they only offer repeated peak dissipation when they yielded further than a previous cycle. Subsequent smaller cycles, the bulk of an earthquake input, thus receive minimal or no dissipation. The work of Bradley *et al.* [5] also demonstrated that their steel yielding devices were prone to buckle, thereby reducing the axial load and energy dissipation capacities [11]. Amaris *et al.* [12] demonstrated that with externally mounted buckling-restrained mild steel bars, slightly improved performance may be obtained; however, their system retained the same fatigue-prone and residual force deficiencies as in Bradley *et al.* [5].

It then becomes apparent that a more robust form of energy dissipation is needed that satisfies several objectives: (i) energy dissipators should not be at risk of low-cycle fatigue bar fracture; (ii) energy dissipators should be ideally located within the beam–column joint region; (iii) to ensure that the structure re-centres, residual forces in the energy dissipator should either re-centre or creep back towards zero over time; and (iv) the cost of devices should be economical compared with conventional (ductile) design solutions.

In response to these objectives, a high force-to-volume (HF2V) lead-extrusion damper was developed [13, 14]. The HF2V damper is a relatively simple device, as illustrated in Figure 1. It consists of a central shaft with a bulge encased in lead. When the shaft moves, the bulge displaces the lead from one side of the bulge to the other and is capable of sustaining a constant force upon yielding, similar to the behaviour of the mild steel energy dissipation devices. These devices have been shown to be relatively insensitive to velocity [14], with a relationship between damper force

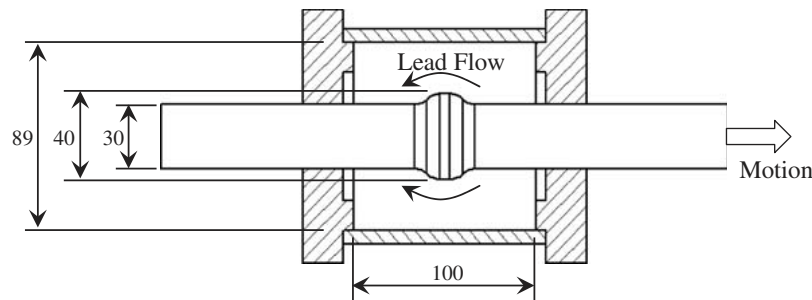


Figure 1. Cross-section of a typical HF2V damper (dimensions in mm).

1 and velocity defined:

$$F = C_{\alpha} v^{\alpha} \quad (1)$$

3 where  $F$  is the damper force;  $C_{\alpha}$  a constant dependent on device architecture;  $v$  the velocity of the shaft; and  $\alpha$  the velocity component, which can be taken as 0.12 [13, 14].

5 The compact HF2V devices are based on the original lead-extrusion dampers developed by Robinson and Greenbank [15]. These early devices were quite large (1–2 m long for 100+ kN capacity), limiting their use to specific applications such as base isolation [16]. In contrast, the unique aspect of the HF2V devices is their ability to fit into tight volumetrically constrained applications, such as directly into beam-column joints [13, 14]. More specifically, 100–350 kN devices have been built that are 120 mm long, with a 100 mm outside diameter of the confining cylinder, considerably smaller than other devices of similar capacity [13].

11 In this study, two HF2V devices were designed and fabricated to be mounted on external anchor plates protruding from each side of the beams. The experimental testing is intended to confirm the benefits of such devices when incorporated into ductile-jointed connections. In addition, results can be compared with the spectral analyses and design guideline predictions developed for these devices [13, 14].

## 17 DESIGN DETAILS

### *Subassembly development*

19 A 3D subassembly representing an exterior joint on a lower floor of a 10-storey prototype building was developed as shown in Figure 2. The subassembly consisted of two beams cut at their midpoints and an orthogonal beam cut at its midpoint, the approximate location of the point of contraflexure. All of these members frame into a central column. The orthogonal beam, referred to as the gravity beam, was designed for supporting one-way precast flooring panels. The other two beams, referred to as the seismic beams, were designed for predominantly seismic forces. The dimensions of the prototype members were taken from previous research [17]. Specific dimensions include: 850 mm square columns, 700 mm deep by 500 mm wide beams, and a 3.6 m storey height. The prototype joint was assumed to have a moment capacity of 500 kNm.

27 Given these constraints, the subassembly was scaled to 80% of the prototype building. The column was scaled to 750 mm square and the beams scaled to 560 mm deep by 400 mm wide.

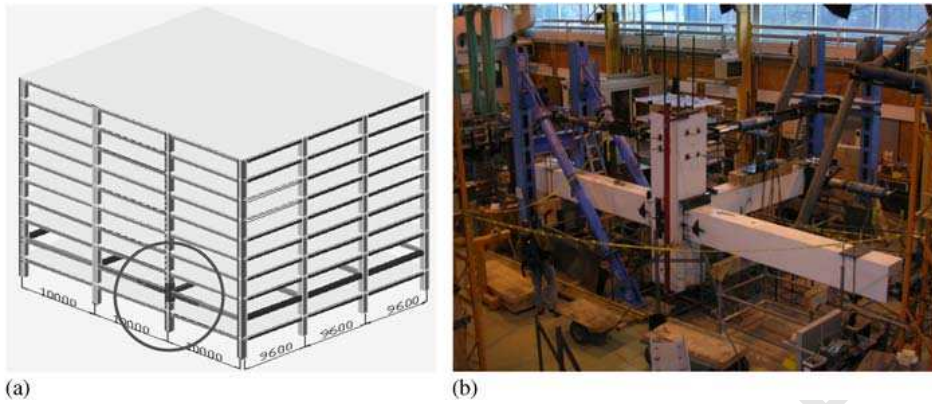


Figure 2. (a) Prototype structure showing location of subassembly [17] and (b) photograph of experimental set-up.

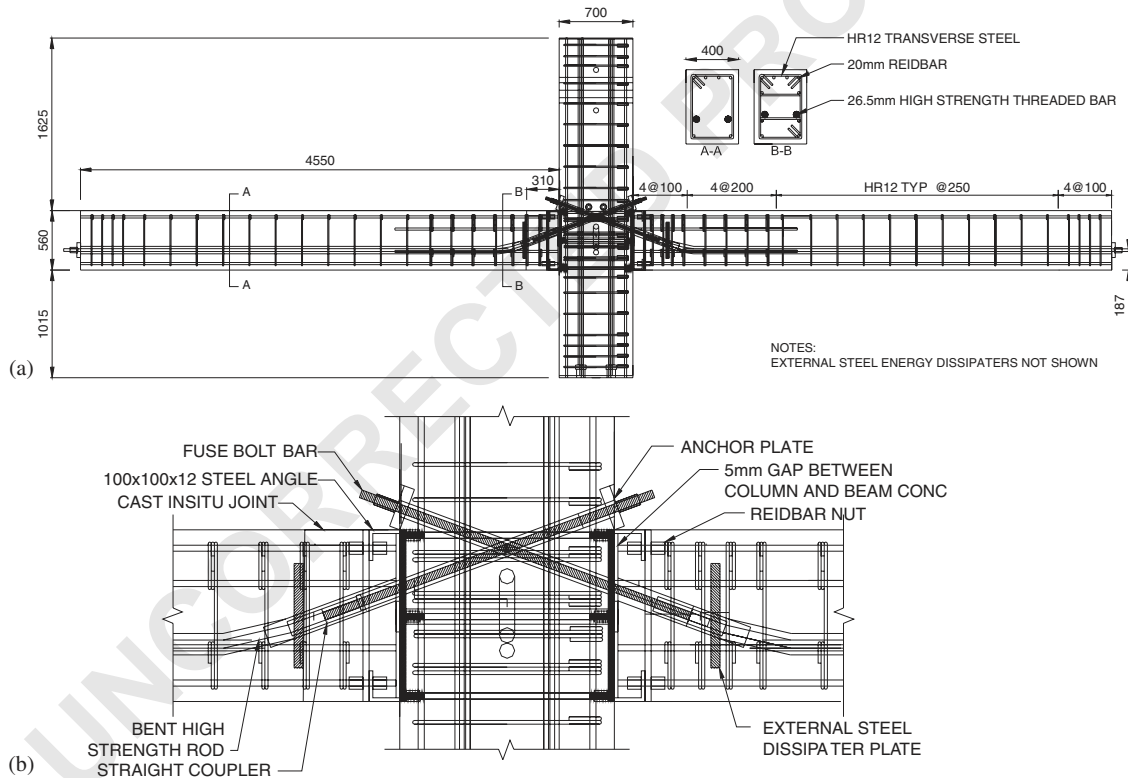


Figure 3. (a) Elevation of the seismic beams and column and (b) details of the seismic beam–column joint experimental subassembly.

- 1 Figure 3(a) shows the seismic beams, column and basic dimensions. Minimal longitudinal reinforcement (ratio of 0.01) was provided for the column and the beams. Figure 3(b) illustrates the basic reinforcing layout for the beams and the column.
- 3

1 The reinforcement arrangements were designed for the expected strength of the rocking  
2 joint from the dissipation devices and prestressing and were intended to remain elastic. Four  
3 D20 longitudinal threaded rebars (Reidbar<sup>TM</sup>,  $f_y = 500$  MPa) were provided top and bottom,  
4 providing the required moment capacity of  $\phi M_n = 260$  kNm. Owing to the presence of axial load  
5 from prestress, minimal transverse steel requirements governed the design; thus 12 mm diameter  
6 (HR12,  $f_y = 500$  MPa) stirrups were provided in the beam at a spacing of  $d/2$  and a closer spacing  
7 at the ends. Additional transverse reinforcement was provided at the top and bottom, 1.2 m from  
8 the beam ends, to confine the concrete in these high-compression zones.

9 Unbonded post-tensioned prestressing was provided by two 26.5 mm diameter high-strength  
10 thread bars placed in 50 mm PVC ducts, each post-tensioned to 200 kN, giving 400 kN of total  
11 prestress to each seismic beam, as illustrated in Figure 3(b). A detailed diagram of the seismic  
12 beam-column joint is given in Figure 3(b). The 'seismic' beams utilized a straight coupler system  
13 where the tendons were pre-bent at the joint end to a radius of approximately 1.8 m, allowing  
14 proper alignment with the angled rod running through the column. The fuse bolt bar was machined  
15 to 75% of its effective area to ensure that any yielding in the prestress system would be limited  
16 to the replaceable through-column bolt bar. The tendon detailing resulted in an eccentricity of  
17 190 mm from the top rocking edge and 370 mm from the bottom rocking edge. At the beam end,  
18 a  $100 \times 100 \times 12$  inverted steel angle was used at the top and bottom of the joint and the face  
19 of concrete was recessed 5 mm. This design ensured that contact with the column was limited  
20 to the steel and allowed the angle's buried flange to mechanically develop stress in the beam's  
21 longitudinal rebars using Reidbar<sup>TM</sup> nuts.

22 By the nature of precast concrete and rocking connections, it is critical that the face of the beam  
23 be aligned flush with the column. Therefore, offsite erection of a full length beam section may  
24 lead to on-site misalignment issues, which may affect rocking behaviour. To mitigate potential  
25 misalignment and also to allow for construction tolerances similar to current standards, a 310 mm  
26 cast *in situ* closure pour was provided on the west seismic beam. Such a closure pour is expected  
27 to be cast on-site after the armouring angles have been adjusted to ensure a flush face at both ends  
28 and the post-tensioning rods are coupled together.

29 High-strength, fibre-reinforced concrete using 2% Dramix<sup>TM</sup> steel fibres by weight was used  
30 in the *in situ* end to compare its behaviour with the regular strength concrete of the east beam.  
31 The mix was designed for high strength and good workability. The measured compressive cylinder  
32 strength of the high-strength concrete was  $f'_c = 70$  MPa, whereas the east beam and the remainder  
33 of the west beam concrete was measured as  $f'_c = 37$  MPa.

34 At each joint, four 30 mm diameter shear keys were installed, tapered  $5^\circ$  inwards to ensure that  
35 they do not jam when the specimen rocks at the beam-column joint. These were designed to be  
36 screwed into the face of the column via a cast in double nut. The shear keys were designed to resist  
37 gravity and seismic shear forces. One shear key was located in each corner, providing torsional  
38 resistance.

### 39 *The HF2V devices*

40 HF2V devices were designed to be mounted externally on purpose-built plates in the beams. An  
41 assembly of the HF2V dampers is given in Figure 4. Device design and characterization details  
42 can be found in Rodgers *et al.* [13, 14]. A central shaft with a streamlined bulge was encased in  
43 a cylinder filled with lead. Rectangular end caps were bolted together via two attachment rods,  
44 which also anchored the device to the beam's anchor plate. One end of the central shaft was

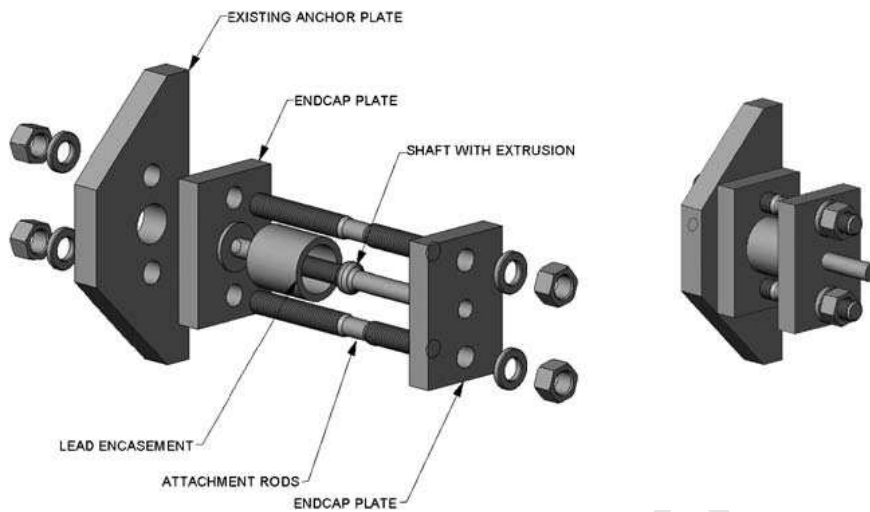


Figure 4. The HF2V damper externally mounted on the beam's anchor plate.

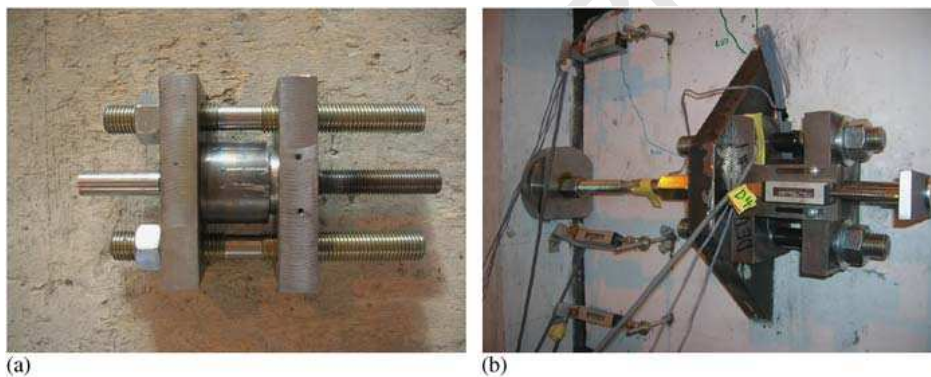


Figure 5. Photographs of the external HF2V damper: (a) prior to installation and (b) mounted on the east beam.

1 threaded, allowing it to be coupled to a threaded rod anchored to the column. The threaded rod  
 3 running through the column was anchored on each end of the column face using a nut and steel  
 plate washer. A photograph of the device and the mounting configuration is given in Figures 5(a)  
 and (b), respectively. The force capacity of these devices is 125–130 kN.

5 Device force capacity affects the mechanics and behaviour of the joint. More specifically, to  
 ensure that the connection is capable of closing, the moment contribution from the initial prestress  
 7 force must exceed the contribution from the dampers. This criterion can be written as follows:

$$\phi M_{PS,i} \geq \Omega_{diss} M_{diss} \quad (2)$$

9 where  $M_{PS,i}$  is the moment contribution at the joint from the initial prestress force;  $M_{diss}$  the  
 moment contribution at the joint from the energy dissipation devices in compression;  $\Omega_{diss}$  the over-  
 11 strength factor of the dissipation devices (taken as 1.5);  $\phi$  the undercapacity factor for the prestress

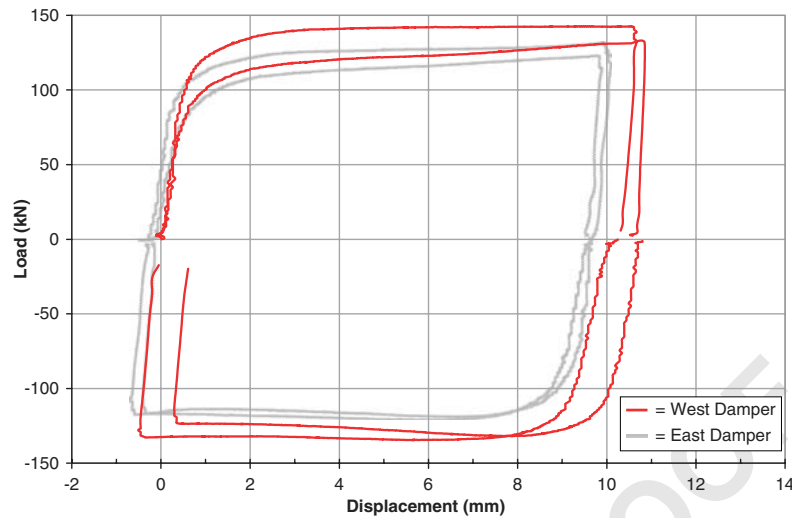


Figure 6. Force–displacement response of the HF2V dampers prior to testing.

- 1 (taken as 0.85). Dividing the contribution from prestress by the contribution from the dissipation  
 2 devices gives a ratio of their expected contribution:

$$3 \quad \lambda = \frac{\phi M_{PS,i}}{\Omega_{diss} M_{diss}} \quad (3)$$

4 To ensure that the system re-centres,  $\lambda \geq 1$  must hold true. The dampers were therefore specifically  
 5 designed [13] to provide the same level of energy dissipation as mild steel devices adopted by  
 6 Li *et al.* [17], corresponding to a designed 120 kN yield force. This design force corresponds to  
 7  $\lambda = 1.3$  and 2.5 in the east–west direction for positive and negative moments, respectively, which  
 8 reflects the eccentricity of the tendons at the joint. As two seismic beams meet at the one column,  
 9 the overall joint is symmetrical as the eccentricity of the tendon profile is counteracted when the  
 10 opposing beam is opening in the opposite direction, giving a ratio of  $\lambda = 1.9$ .

11 To validate the as-built devices, reversed cyclic quasi-static (QS) tests were performed on the  
 12 HF2V dampers using a uni-axial compression–tension testing machine. The experimental force–  
 13 displacement response of the dampers subjected to these tests is shown in Figure 6. The devices  
 14 exhibit a clear elasto-plastic hysteretic behaviour, with an average initial stiffness of 200 kN/mm.  
 15 The average force capacity is at the design level, with forces of 120–130 kN.

## EXPERIMENTAL SET-UP AND METHODS

17 Figure 7 shows the overall experimental test set-up including (a) a plan view and (b) a south  
 18 elevation view. As shown in Figure 7, loads were applied to the specimen by three hydraulic  
 19 actuators: actuators A and B were installed on the reaction frame and top of the east and south  
 20 face of the column, respectively, and actuator C was installed in the east–west direction at the  
 21 end of the gravity beam. Actuator C was intended to stabilize the specimen, with its movement  
 synchronized to one-half of the displacement of actuator A. Load cells were installed in series

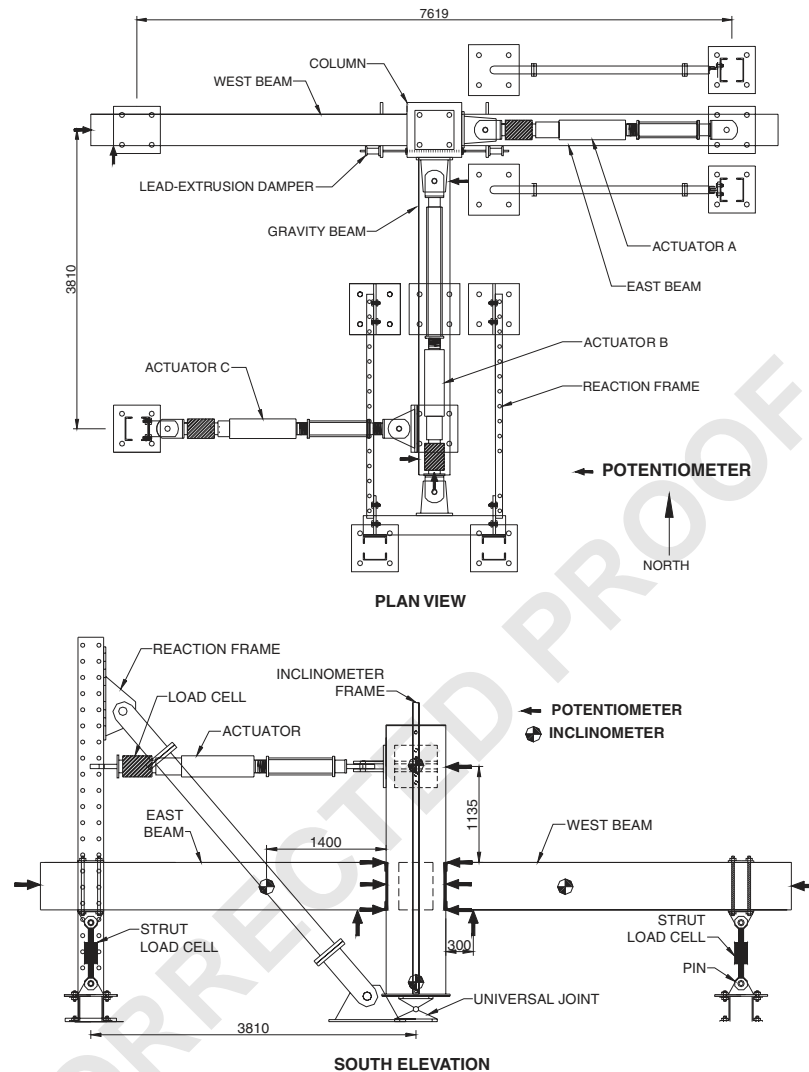


Figure 7. Schematic diagrams of the test set-up and instrumentation.

- 1 with each actuator. Additional load cells were attached at the strut of each beam, the jacking point
- of each post-tensioned rod, and on the connecting rods to the HF2V devices.
- 3 To measure the rotation at the joint, three linear potentiometers were installed on both faces of
- 5 each joint, totalling 18 potentiometers. Two additional linear potentiometers were installed against
- the bottom face of each beam to measure vertical movement. At several locations around the
- 7 specimen rotary potentiometers were installed to measure the local displacement. Two 5 mm strain
- gauges were installed on each bolt bar to measure any potential yielding that may occur during
- 9 testing. To measure the response of the HF2V dampers, a load cell and strain gauges were attached
- to the anchor rod of each device. A spring potentiometer was mounted on the back of each device
- to measure the displacement of its central shaft.



1 The instrumentation was installed and calibrated to use 70–85% of the total sensor range  
 3 during testing to 4% column drift. Using data acquisition with  $\pm 2048$  counts, this approach gave  
 5 approximately 3000–3500 counts for 4% drift, giving a resolution of approximately  $\pm 0.1$  kN on the  
 7 actuator load cells,  $\pm 0.05$  kN on the device load cells,  $\pm 0.15$  mm on the large rotary potentiometers,  
 and a theoretical resolution of approximately  $10\ \mu\text{m}$  on the smaller linear potentiometers. The  
 control system operated with a typical deadband of 1 count, equating to  $\pm 0.15$  mm on the lateral  
 column displacement.

9 The testing regime was relatively straightforward and focused entirely on the performance of  
 the HF2V dampers for different design levels of storey drift. Cyclic QS tests were performed  
 at varying levels of storey drift. These tests were uni-directional in the east–west direction. The  
 11 north–south direction is largely disregarded in this investigation, since its performance has already  
 been reported in previous studies [5, 18]. The testing was conducted in two phases, initially using a  
 13 single damper attached to the south side of each seismic beam. The second phase involved removing  
 the east beam and both dampers were placed on the west beam, thus doubling its damping capacity  
 15 (giving re-centring ratios  $\lambda^+ = 0.65$ ;  $\lambda^- = 1.25$ ). The second configuration represents a corner joint  
 of the prototype building.

17

## EXPERIMENTAL RESULTS

*Exterior joint*

19 Experimental results from this section focus on the global performance of the specimen and the  
 local performance of the HF2V dampers. Since the local behaviour of the specimen (i.e. cracking,  
 21 crushing, prestress, etc.) has been reported previously [5, 18], these aspects of the results are not  
 reported here.

23 For purposes of comparison, Figure 8 presents the results of QS testing in the east–west direction  
 to a maximum column drift of 2% for three cases: (i) prestress only; (ii) mild steel energy  
 dissipators; and (iii) the HF2V devices.

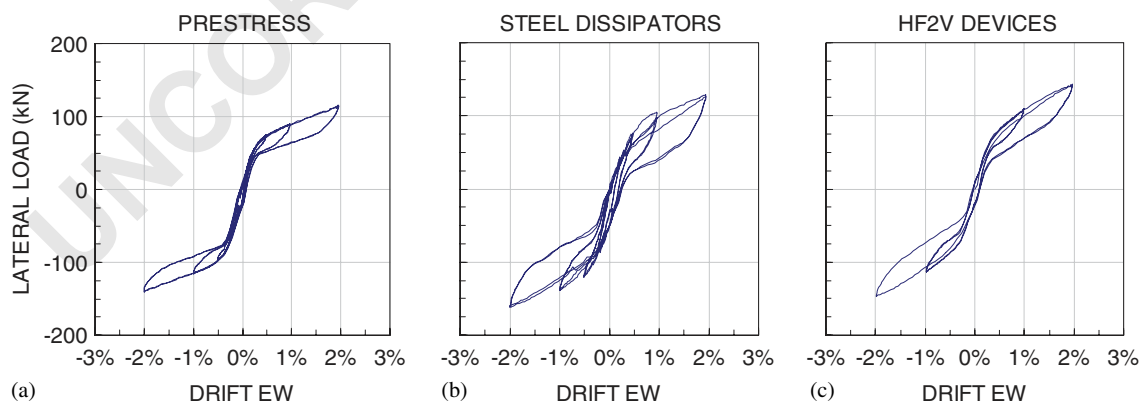


Figure 8. Comparison of the performance considering: (a) prestress only;  
 (b) steel dissipators; and (c) HF2V devices.

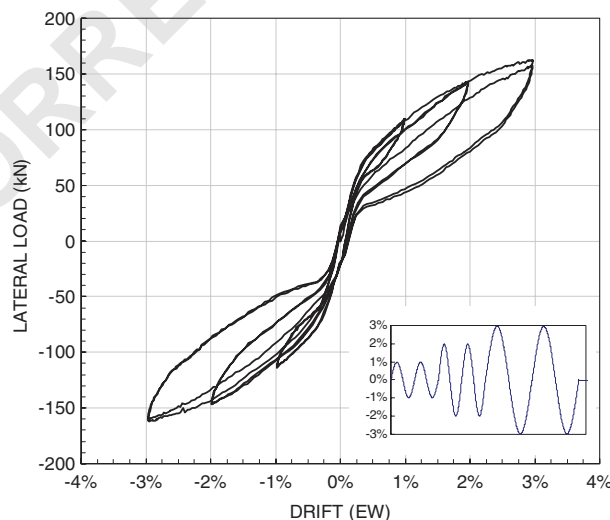
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1 With prestress only, it is apparent that the system still provides some level of energy dissipation,  
 3 largely due to the frictional effects under movement reversal arising from the draped and bent  
 5 thread bars within the ducts. The steel devices provide the most hysteretic energy dissipation for  
 7 these small drift cycles, and the HF2V devices provide some additional damping. The specimen  
 with steel devices exhibits some strength degradation. This loss can be traced to the fact that the  
 steel dissipators will undergo plastic deformation in tension and will not recover that deformation  
 in compression in part due to buckling.

9 Partial buckling of the dissipators also means that less dissipation occurs on subsequent loading  
 11 cycles, potentially leading to larger earthquake-induced displacements. More explicitly, only peak  
 13 cycles receive full dissipation with the mild steel dampers, rather than every cycle regardless of  
 size. The HF2V devices do not exhibit this effect. The hysteresis loops of the specimen that utilizes  
 the HF2V devices are stable, showing no apparent strength degradation on the second cycle of  
 loading for a given drift amplitude.

15 On larger displacement cycles, the HF2V dampers provided a significantly greater amount of  
 17 hysteretic energy dissipation. This result can be seen in Figure 9, where an additional two cycles  
 to 3% drift reveal a markedly large change in response. The specimen exhibited some drop in  
 strength during reloading to the previous peak, but no overall capacity was lost.

19 This loss can be seen more clearly in the force–displacement response during testing of the  
 21 HF2V devices, which is shown in Figure 10. Before the specimen reached a drift of 3%, the devices  
 23 behaved essentially in an elastic manner. At 3% drift, the devices fully engaged and yielded,  
 with the shaft moving approximately 3.5 mm. Upon subsequent loading and unloading, the device  
 exhibits an ‘elasto-plastic’ type of response, with slight ‘pinching’ at zero force. This pinching can  
 be attributed to take-up within the connecting threaded elements of the device. The shaft does not  
 return to its initial position, but upon completion of the test remains about 1.5 mm from its initial  
 position, due to take-up and mount flexibility.



25 Figure 9. Response of the specimen with HF2V dampers to 3% drift  
 (inset: quasi-static loading regime).

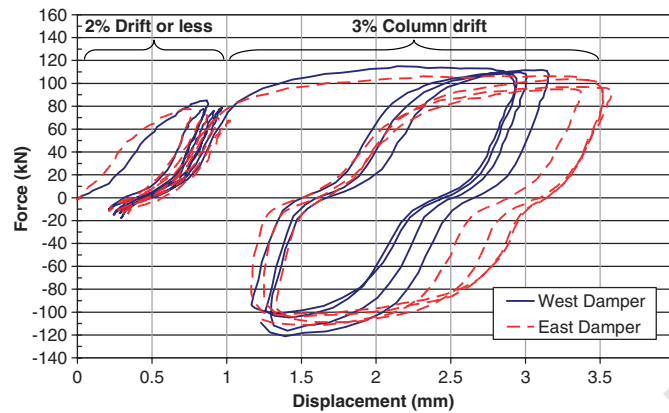


Figure 10. In-service force–displacement response of the HF2V devices.

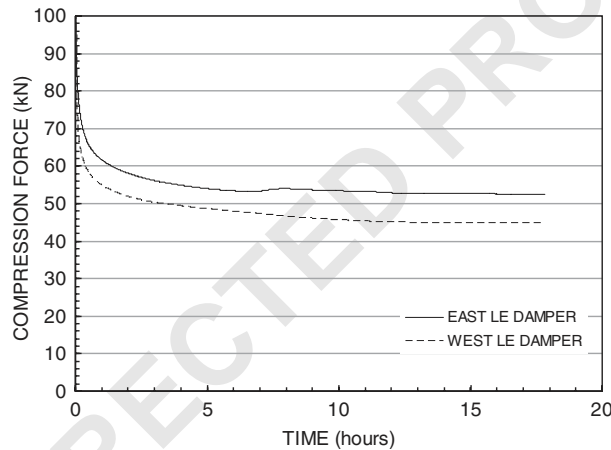


Figure 11. Damper force over time.

1 After testing, the residual compression force in the HF2V damper and anchor shaft was approx-  
 3 imately 90 kN. A logarithmic decrement of force occurred over time, as shown in Figure 11.  
 5 Within the first 5 h the force in the devices had dropped by some 50 kN—about 45% of the peak  
 7 compression force. This is a clear advantage of the lead-extrusion dampers over the steel fuse bars.  
 More specifically, as well as remaining completely damage-free and operating efficiently for every  
 cycle, the residual compressive force creeps out of the devices, leading to complete self-centring  
 of the joint. Note that the low residual forces in Figure 11 are well within the ( $\lambda > 1.0$ ) re-centring  
 capability of the joint, as well.

#### 9 Corner joint

11 As noted in the previous section, after the initial testing, the east beam was removed and its damper  
 was relocated to the west beam, doubling the beam's energy dissipation capacity. Figure 12 presents  
 the response of the specimen with and without the dampers, subjected to fully reversed loading

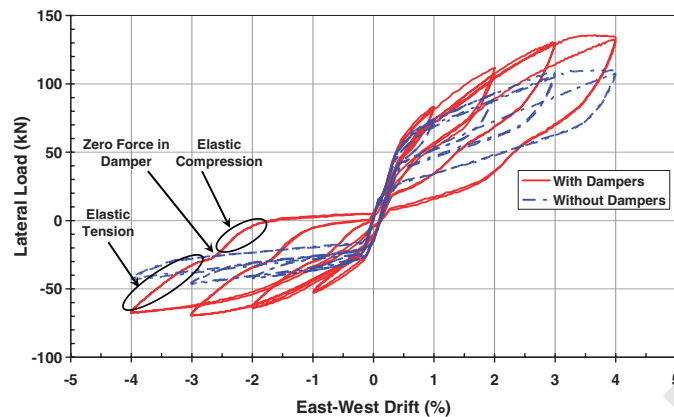


Figure 12. Response of corner joint specimen QS testing to 4% drift.

1 cycle out to 4% drift. The asymmetry of the force–drift response is due to the eccentricity of the  
 2 tendon with respect to the beam mid-depth. Although this asymmetry has always been present,  
 3 the hysteresis loops presented in Figures 8 and 9 do not show this asymmetry, as the presence of  
 4 both the east and west beams balanced out this effect. More specifically, although the forces at  
 5 each interface were asymmetrical, the west interface was undergoing the opposite joint rotation to  
 6 the east interface, resulting in overall symmetry of the hysteresis loops. The removal of the east  
 7 beam removed this cancellation, resulting in the asymmetry seen in Figure 12.

8 It is apparent that the specimen exhibits significantly more energy dissipation when compared  
 9 with previous tests in Figures 8 and 9, as was expected due to the increased damping forces.  
 10 However, re-centring was still apparent despite  $\lambda < 1.0$  for negative direction loading, with smaller  
 11 tendon eccentricity. Finally, the residual displacements were negligible.

12 Some pinching, indicated as localized ‘flattening’ in the hysteresis loop, was observed during  
 13 the unloading phase. This phenomenon is attributed to minor take-up within the threaded anchor  
 14 rods, which occurs when the rod forces change from tension to compression. This phenomenon  
 15 was confirmed by the load cell readings, indicating that this behaviour coincided with zero load  
 16 in the devices.

17 Interestingly, the area enclosed within the hysteresis loops for the joint without the dampers  
 18 shows a large disparity between the two directions. Again, this phenomenon can be traced back  
 19 to the tendon profile. The inherent hysteresis for the joint without dampers is primarily related  
 20 to the well-known friction effects that exist in draped tendons, in this case between the prestressing  
 21 tendons and the PVC ducts in which they are contained. The bent tendon profile results in notable  
 22 friction between the tendon and PVC ducts, as the tendon undergoes deformation during gap  
 23 opening. The drape (curve) of the tendon naturally leads to higher friction for gap opening in one  
 24 direction than the other.

25 Under positive loading when the drifts exceeded +3.4%, some minor strength degradation was  
 26 observed as a result of yielding of the reduced-section thread bars. This is not considered to be  
 27 a bad feature, rather it was intended, by design. If substantial yielding occurs when drift exceeds  
 28 say 5%, then the connecting reduced-section thread-bars can be replaced.

29 Finally, to quantify the relative amounts of damping provided by the jointed connection, without  
 any supplemental damping devices, with yielding steel fuse bars, and with the HF2V dampers,

Table I. Energy dissipated for different joint configurations (J).

	Drift level (%)					
	0.25	0.5	1	2	3	4
External joint configuration						
Post-tensioning only	67	141	420	1208	2372	—
Steel fuse bars	114	331	819	2200	—	—
HF2V dampers	—	—	440	1601	3419	—
Corner joint configuration						
Post-tensioning only	—	—	223	666	1297	2007
HF2V dampers	—	—	247	946	2380	4230

*Note:* The steel fuse bars are not tested in the corner joint configuration, and the HF2V dampers are at a minimum capacity for the size of the connection for the external joint set-up.

1 the areas enclosed within the hysteresis loops were numerically integrated. These areas represent  
 3 the energy dissipated. The results of this integration are presented in Table I, where the values  
 3 represent the average enclosed area across four peaks to each drift level, resulting from two fully  
 reversed sinusoidal loading patterns.

5 It should be noted that since the figures are an average over four cycles, the values do not  
 7 necessarily represent a stable and repeatable amount of energy dissipation, and the large dissipation  
 7 on the first cycle may not be matched on subsequent cycles. In particular, at 2% drift levels, an  
 9 overall average of 2200J is dissipated by the joint with steel fuses, compared with 1600J for  
 9 the joint with HF2V dampers. However, the much larger dissipation for the steel is only due to  
 11 the large dissipation on the initial cycles, with the third and fourth cycles having dissipation of  
 11 approximately 1800J, bringing it much closer to the energy dissipated by the joint with the HF2V  
 dampers. Moreover, the steel fuses were not tested in uni-directional testing up to 3 or 4% drift;  
 13 hence, comparisons could not be made at larger column drifts where the HF2V dampers provided  
 better dissipation.

15 The larger dissipation from the HF2V dampers at higher drifts can be attributed to the take-up  
 or slackness in the connecting elements. The corner joint results show that at 1% column drift,  
 17 the HF2V dampers increase the inherent hysteretic damping of the joint by approximately 11%,  
 17 whereas the increase over the undamped joint becomes 42, 84, and 110% at column drifts of  
 19 2, 3, and 4%, respectively. The lack of contribution at the lower drift levels could be mitigated by  
 careful design of the connecting elements to eliminate flexibility and take-up/slackness.

21 Overall, the energy dissipation provided by the HF2V dampers is somewhat underrepresented  
 due to both the conservative size of the dampers chosen and the sources of flexibility. Much  
 23 larger dampers could be provided in the external joint configuration while maintaining static joint  
 re-centring ability, as demonstrated by the corner joint configuration. Hence, the highly repeatable  
 25 (cycle-to-cycle) HF2V device results could be readily improved by increasing the device force  
 capacity, thus presenting a different comparison to the steel fuse bars than that seen in Table I.

## DISCUSSION

27  
 29 The performance of the specimen met the two primary performance objectives: (i) the dissipation  
 of considerable energy by the supplemental HF2V devices and (ii) with no structural damage.

1 This proof-of-concept experiment has validated the efficacy of incorporating HF2V devices into a  
2 beam–column joint to reliably dissipate seismic energy. Although the devices were slow to ‘engage’  
3 and take full effect at small drifts due to some mount flexibility, at higher drifts it was found that  
4 the specimen exhibited much larger hysteresis loops and therefore absorbed considerably more  
5 energy than with the mild steel devices, which buckled on compression. The flexibility issue could  
6 be easily mitigated in design by selecting a larger shaft diameter and/or connecting rods with  
7 lower levels of stress during operation. The ability of the extrusion-based devices to dissipate the  
8 residual compressive force over time through creep effects means that essentially zero moment  
9 exists at zero drift ensuring full re-centring capability of the structure. Moreover, no maintenance  
10 of the damping system following an earthquake is expected to be needed.

11 As noted previously, the devices were slow to ‘engage’, as evidenced by the stark difference  
12 in hysteretic energy dissipation shown in Figures 8 and 9, between 2 and 3% column drift. This  
13 lag was traced (in part) to the anchoring system utilized. Given the properties of the anchor rod,  
14 its elongation at the yield force of the damper (120 kN) would be approximately 2.5 mm. Given a  
15 column drift of 2%, which corresponds to a connection rotation of about 0.015 rad, the maximum  
16 expected gap opening at the location of the extrusion dampers is 4.5 mm. The elongation of the  
17 rod would account for more than half this amount, thus seriously reducing the effectiveness of the  
18 device. This effect is magnified by the fact that each connecting element (the coupler, nuts, anchor  
19 plates) exhibits some degree of take-up, which must also be accommodated before the full yield  
20 force of the dissipator can be reached. Given the elastic stiffness of the damper (200 kN/mm),  
21 such effects can be notably detrimental to the full effectiveness of the dampers’ contribution to the  
22 joint. This effect can be best further explained looking at the system based on strain energy. For  
23 maximum energy dissipation, it is desirable to have the minimum amount of elastic strain energy  
24 in the connecting elements, so that the largest displacement is applied directly to the dampers,  
25 which will in turn lead to the greatest effectiveness of the HF2V devices in absorbing energy.

26 The lag effect of damper mount and connecting rod flexibility is also the reason why the  
27 specimen exhibited minor strength degradation on secondary cycles. At the completion of a cycle,  
28 the damper and its connecting elements would be in compression. At the onset of a reversed cycle,  
29 the anchor rods need to first transition from a fully elastic compression state to an elongation in  
30 tension, thus requiring even greater displacements of the joint for yielding of the dissipation device  
31 to occur. This effect may be considerably reduced if the damper and its anchorage elements are  
32 prestressed, thus eliminating any sources of flexibility and take-up in the system. Other solutions  
33 include lower stresses in all elements connecting to the HF2V devices, if warranted, or embedded  
34 devices within the concrete members.

35 In future efforts, care should be taken when designing the dissipator anchorage, with specific  
36 attention given to eliminating any slackness or take-up within the system. During initial testing  
37 of the extrusion dampers (prior to mounting them on the specimen) and from previous published  
38 work [16] it was found that the peak yield force of the devices was weakly velocity dependent,  
39 with a velocity exponent in the range of 0.11–0.15. This effect may be beneficial, as under very  
40 rapid displacements, such as would be expected in a large earthquake, the device would provide  
41 additional damping to the structure. This allows the dampers to provide a force higher than the  
42 restoring force from prestress, while still allowing the connection to re-centre upon unloading,  
43 unlike sacrificial steel-yielding bars/devices. However, if the increase is significant, it is possible  
44 that the increased force is detrimental to the weak beam–strong column, capacity design principles  
45 utilized here. Therefore, the damper velocity characteristics should be utilized to predict force  
46 levels for the dampers under full dynamic loading, and these forces incorporated into the capacity

1 design procedures. In particular, careful design can account for, and eliminate, any detrimental  
 3 effects relating to this velocity dependence. More specifically, careful damper design, based on  
 well-defined design equations [13], along with careful connection design, will readily provide an  
 optimal outcome.

## 5 CONCLUSIONS

7 This study has presented a proof-of-concept experimental investigation. Compact high force-to-  
 volume (HF2V) devices capable of being fitted directly into a structural connection, an alternative  
 to mild steel energy dissipation devices, were mounted externally across a beam-column joint.  
 9 Based on the results of this study, the following conclusions can be drawn:

- 11 1. It was demonstrated through QS testing that the HF2V dampers could provide a level of  
 energy dissipation comparable to, or in excess of, mild steel devices designed for the same  
 yield force at low drifts. Larger drifts saw significantly increased energy dissipation from  
 13 the extrusion dampers. Subsequent smaller motions received full dissipation, which would  
 not occur with yielding dissipators.
- 15 2. HF2V extrusion dampers offer an attractive alternative to mild steel energy dissipation  
 devices. These devices do not suffer from low-cycle fatigue and the force in the devices  
 17 creeps back towards zero upon unloading. Therefore, they do not need any maintenance  
 following an earthquake and could be mounted internally. The ability of the device force  
 19 to creep out over time ensures that self-centring of the structure is maintained following an  
 earthquake.
- 21 3. The stiffness of device mounts and connecting elements is an important consideration. Since  
 gap opening is generally 5–10 mm, stiffness plays an important role in the effectiveness of  
 23 supplemental energy dissipation. Care should be taken when designing the anchorage for  
 these devices. Prestressing the devices will help to eliminate take-up in the system and ensure  
 25 a more optimal solution.

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