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Development of the Self-Centering Sliding Hinge Joint

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ABSTRACT: The Sliding Hinge Joint is a beam-column connection used in low damage moment-resisting steel frames. It allows inelastic deformation with small but significant losses of elastic strength and stiffness during a major earthquake, and does not always return the joint and overall building to the pre-earthquake position. It thus does not fulfil the optimum requirement of no maintenance. This paper presents the ongoing development of a damage free self-centering Sliding Hinge Joint utilising friction Ring Springs. This work has commenced with tests to determine (1) the effects of steel shims of different hardness on the sliding behaviour, (2) the adequacy of the bolt model previously developed for the SHJ, and (3) the residual joint strength of the post-earthquake joint. The abrasion resistant Grade 400 plate, the hardest steel considered, generated the largest sliding shear capacity, and the most stable sliding characteristics. It is therefore recommended for use in future SHJ construction. The bolt capacities and residual joint strengths are affected by bolt size, length of bolt lever arm and presence of Belleville Springs. These effects are not yet fully understood at the time of writing. More tests of various specimen sizes will be used to develop a more accurate model.

1 INTRODUCTION

The conventional rigid welded beam to column connections in moment resisting steel frames (MRSFs) suffered premature brittle fracture in the beam flange to column flange welds in the 1994 Northridge and 1995 Kobe earthquakes. Alternative strategies allowing controlled damage have since been developed to avoid weld failure in MRSF beam to column connections. This usually involves forcing plastic hinges to form in the beam away from the column face, thereby dissipating energy through ductile deformation in the beams. While effective in providing safety and preventing collapse, these systems are associated with irrecoverable plastic deformation. This can cause heavy economic losses due to post-disaster repair costs and consequences of temporary building closure, particularly in highly developed regions. There has therefore been a shift in worldwide trend to the development of low damage systems. The objective of these is to not only prevent building collapse, but to enable restoration of full functionality following a major earthquake with minimal repair required.

The Sliding Hinge Joint (SHJ) is a key component of one such system used in MRSFs. It was initially developed by the New Zealand Heavy Engineering Research Association (HERA) and the University of Auckland (Clifton 2005) and further developed by the University of Canterbury (MacRae, Clifton et al. 2010). It is an Asymmetric Friction Connection (AFC) which can undergo large inelastic rotations with limited damage, making it suitable for ductile design. This is achieved by pinning the top corner of the beam to the column through the top flange plate, with slotted bolted connections in the bottom web and beam bottom flange to allow rotation (Fig 1a). The bolt groups are designed to be rigid under working load and serviceability limit state (SLS) conditions, slide under Ultimate Limit State (ULS) events while dissipating energy through friction, and return to an effectively rigid connection at the end of the earthquake shaking. As the inelastic action is limited to the joint rather than the typical elasto-plastic beam hinging, the SHJ has many advantages over traditional rigid column beam joints.

However, finite element analysis and experimental testing (Clifton 2005) showed that once the SHJ is forced into the sliding state it undergoes a small but significant loss of elastic strength and stiffness, thus not fulfilling the optimum performance of a return to full rigid strength and stiffness. This occurs as the Property Class 8.8 bolts, which are fully tensioned on installation beyond their yield point, are subjected to additional plastic deformation during the sliding phase. This is caused by the interaction of moment, shear and axial forces generated during sliding, and leads to a drop in bolt tension. The recommended maintenance procedure is bolt replacement, with Belleville Springs installed under the nut. Furthermore, numerical integration time-history (NITH) analyses on representative frames incorporating the SHJ do not fully return to their pre-earthquake position at all instances. This paper describes the next stage of SHJ development being undertaken, ie:

1. Concept and developments of a fully damage-free self-centering Sliding Hinge Joint (SCSHJ).
2. Tests determining the effects of steel shims of different hardness on the sliding behaviour.
3. Tests determining the adequacy of the bolt model previously developed for the SHJ, and the residual joint strength of the post-earthquake joint.

2 SELF-CENTERING SLIDING HINGE JOINT DEVELOPMENT

A concept for a self-centering SHJ utilising four Ring Springs per joint was proposed by Clifton (2005) and its effectiveness in retaining post earthquake strength and stiffness was demonstrated theoretically through Numerical Integration Time History (NITH) analyses. However no physical testing was undertaken due to a lack of time and resources, and the impracticality and high cost of four Ring Springs (RS) required per joint. RS are friction damping springs produced by Ringfeder, Germany (Ringfeder Gmbh 2008). They have high load capacities and can be prestressed up to 50% of their maximum capacity without loss over time. Their compression capacity and deformation capacity can be adjusted independently. Their characteristics are independent of loading rate, temperature and humidity, making them ideal for use in the SCSHJ. While there are no current applications of the springs in beam column joints, pre-stressed RS have been used in the column base of the Victoria University student accommodation seismic resisting system (Gledhill, Sidwell et al. 2008) to provide an effective low damage concentrically braced frame seismic-resisting system.

2.1 SCSHJ layout and mechanics

The SCSHJ concept presented here combines the SHJ behaviour and the energy storing properties of RS connected to the beam bottom flange and column flange. It is simpler and lower cost than the concept proposed by Clifton (2005), with only one or at most two RS required for each joint. This is enabled by maximising spring efficiency by detailing that ensures the RS deforms in compression regardless of the loading direction. The layout and RS detailing is shown in Figure 1a.

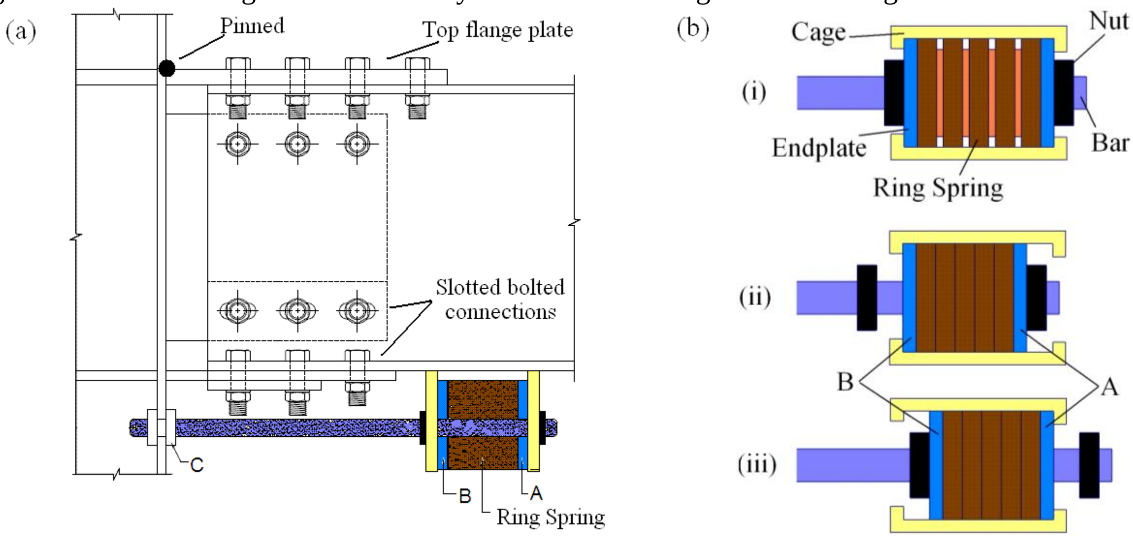

Figure 1: (a) SCSHJ layout (side of cage removed for clarity) and (b) Ring Spring layout (view from below the beam and the cover plate is now shown)

The RS components consist of a bar and nuts, endplates, and cage as described below:

1. The bar is fixed to the column flange, shown as C in Figure 1a, and runs through the RS. It holds and transfers load from the column to the endplates.
2. The endplates at each end of the RS transfers load from the bar and compresses the RS.
3. The cage comprises two side plates with protruding edges and a cover plate underneath the spring. The length of the side plates determines the RS pre-stressing levels when the joint is at rest. The cage transfers load into the beam, and encloses the RS and endplates to guide the spring during compression.

When a positive moment is applied, the joint remains rigid until the load overcomes the combined frictional shear resistance and RS pre-stress levels. The joint then starts to slide as the column rotates away from the beam. This pulls the bar, opening a gap between cover plate A and the cage interface. Cover plate B bears on the other end of the cage, causing the RS to compress. This is shown in Figure 1b(ii). When a negative moment is applied (Fig 6biii), the column pushes the bar towards the beam, opening a gap between cover plate B and the cage, compressing the RS. This is shown in Figure 1b(iii). The RS configuration is similar to a RS seismic damper proposed and tested by Filiatrault, Tremblay et al. (2000). They reported stable and repeatable force-displacement characteristics, with minimal degradation of the damper. Shake table tests and numerical studies on a single storey braced frame showed that the damper effectively dissipated energy and reduced lateral displacements and accelerations.

The RS will improve the SHJ performance in two ways. Firstly, the force generated in the RS contributes to joint moment capacity, and thus the number of bolts in the sliding components can be reduced. While the energy dissipated is reduced, the frictional resistance to joint re-centering also reduces. Secondly, a portion of the energy normally dissipated in the current SHJ is stored in the spring. Upon load removal or reversal, the energy is released and contributes to the restoring force that returns the joint to its original position. Loss in joint strength or stiffness can be compensated by prestressing the RS, which does not undergo degradation during shaking. Figure 2 shows the idealised hysteresis curves for the standard SHJ, the RS component, and resulting SCSHJ. The SCSHJ does not reflect the flag-shaped hysteresis curves typical of most self-centering systems currently under development. The objective of these systems, such as the post-tensioned steel tendon and its variations (Herning, Garlock et al. 2009) and shape memory alloy (SMA) systems (DesRoches, Taftali et al. 2010), is to develop static recentering properties. The objective of the SCSHJ is to improve dynamic recentering properties to a dependable level. An alternative to RS is nitinol SMA springs which are capable of undergoing large recoverable inelastic strain (Desroches, McCormick et al. 2004).

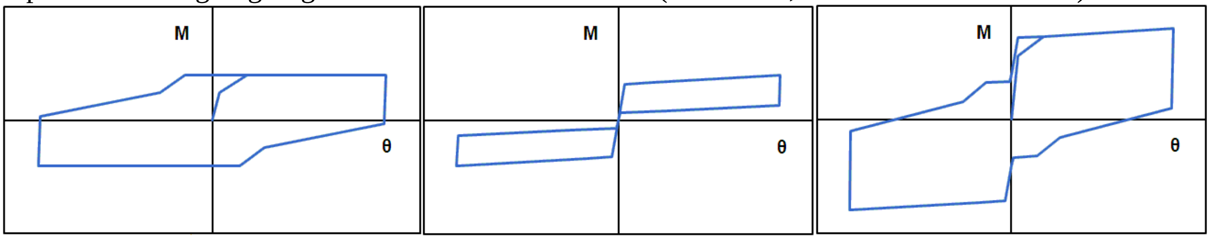

Figure 2: Idealised moment rotational behaviour, left to right: SHJ, Ring Spring, and SCSHJ

3 JOINT COMPONENT TESTS

Component tests were conducted to determine (1) the effects of steel shims of different hardness on the sliding behaviour, (2) the adequacy of the bolt model previously developed for the SHJ, and (3) the residual joint strength of the post-earthquake SHJ.

3.1 *Influence of shim material*

Brass shims were originally tested and recommended by Clifton (2005) to facilitate smooth, stable sliding in the SHJ. This was in accordance with the findings of Popov and Grigorian (1994), who reported rapid fluctuations in friction for symmetric sliding between mild steel surfaces, and smooth and stable friction for mild steel on brass. MacRae et. al (2010) then compared mild steel and brass shims in three-quarter scale beam to column SHJ tests loaded pseudostatically, and found that they both exhibited very similar moment-rotation behaviour and capacity. This is due to the asymmetric sliding nature of the SHJ eliminating the peaks shown by symmetric friction sliding. Mild steel shims are cheaper, more readily available, eliminate dissimilar metal issues at the interface, and can be tack welded in place. They have thus been adopted in recent SHJ building construction. Nevertheless, there were concerns that mild steel shims have yet to be tested dynamically. This series of tests was therefore undertaken to determine the dynamic performance of Grade 300 mild steel (G300) shims, as well as high strength, quenched and tempered Grade 80 steel (G80), and abrasion resistant Grade 400 steel (G400). The critical variable was thought to be the material hardness, with the hardness of G300, G80 and G400 measured to be 168 HB, 266 HB and 382 HB in the Brinell scale respectively. The driver behind these tests was the theory that sliding between steel surfaces of significantly different hardness would exhibit reduced wear and more stable sliding, similar to brass on steel.

The test setup (Fig 3) replicated the rotation of a SHJ with a 460UB beam. The specimens consisted of 16 mm thick cleats, cap plates and beam flange plates. The bolts were four M24, Class 8.8. The thickness of G300, G80, and G400 shims were 3 mm, 4 mm and 5 mm respectively. The specimens were subjected to increasing cycles of displacements at approximately 3.5 mm, 6.9 mm, 10.4 mm and 13.8 mm in amplitude, corresponding to rotations of 0.0075 rad, 0.015 rad, 0.0225 rad and 0.030 rad.

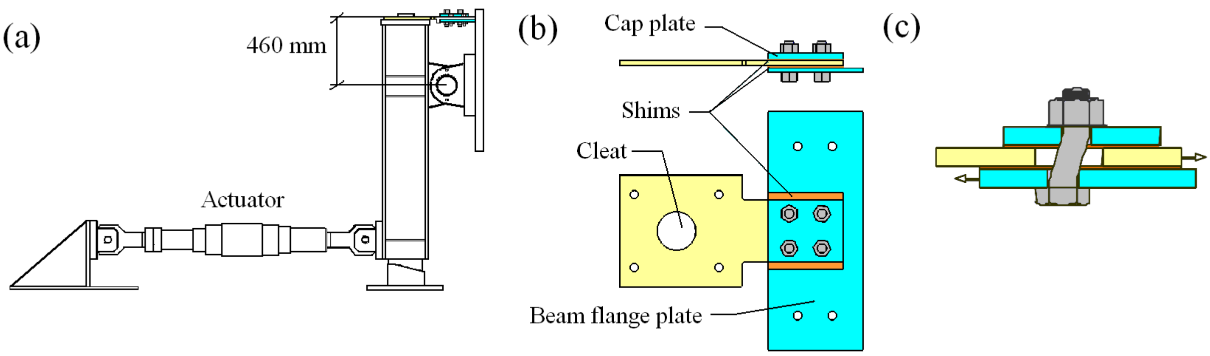

Figure 3: Bolt component test setup (a) layout and (b) side view and plan of specimens, and (c) sliding bolt conditions

3.1.1 *Test results and discussion*

For each material, the sliding behaviour was characterised by an initial stable level of sliding shear at early, low amplitudes of displacement. With larger displacements and further travel, it then increased and stabilised. This is shown in the force-displacement curves for G300 and G400 shims (Fig 4), where A marks the initial sliding shear (V_{ssi}), B marks the stable sliding shear at the final cycle of loading (V_{ssf}), and C marks the maximum sliding shear at any time (V_{ssm}). The material performance was evaluated based on (1) the initial sliding friction force, (2) material wear, (3) stability and repeatability of hysteretic loops, and (4) gain in strength over time. A high initial sliding shear maximises capacity, and a low gain in strength will enable the use of a low overstrength factor in design. Force-displacement behaviour is related to the material wear and should be stable and repeatable to allow the behaviour to be dependably modelled.

G400 produced the most stable loops followed by G80 and G300. It had the highest V_{ssi}, the least wear, and the lowest increase in strength. Table 1 shows the force/bolt, and Figure 5 shows the wear on the cleat and shim surfaces. G300 and G80 shims produced loops that had larger increases between cycles and also increases within the same cycle. This is shown by the larger ratios of maximum to initial sliding shear (V_{ssm}/V_{ssi}), and the maximum sliding shear to that of the final cycle (V_{ssm}/V_{ssf}). The latter ratio shows the increase in shear resistance within the same cycle. The difference in hardness of the surfaces improved the sliding performance. It is therefore recommended that G400 shims be used in future SHJs. The cost is slightly higher, but the sliding shear performance is slightly better than that shown by brass on steel from (Clifton 2005) and G400 has the same availability and fabrication benefits as the G300 shims currently used in practice, while maintaining the same fabrication and construction benefits over brass shims as described above.

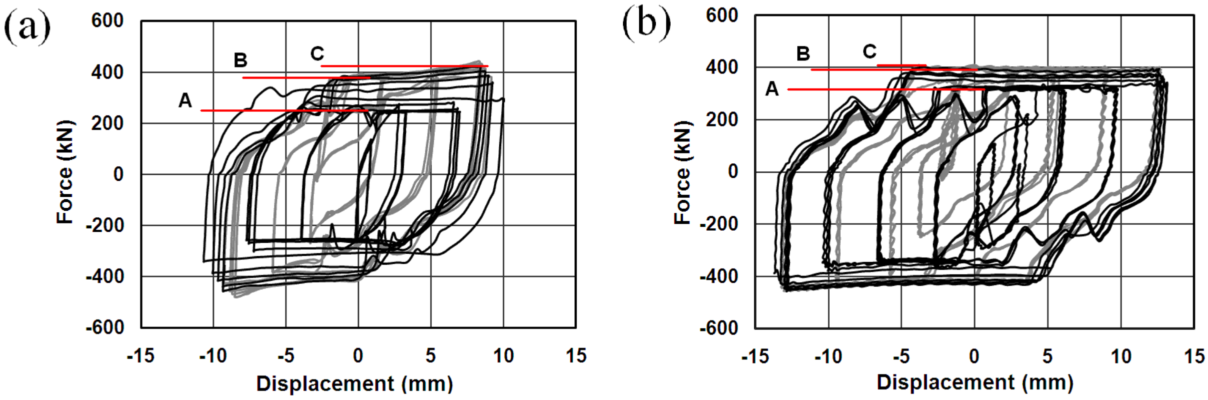

Figure 4: Force displacement curves for (a) G300 and (b) G400 shims

Figure 5: Left to right: surface conditions for cleat with G300 shims, cleat with G400 shims, G300 shim and G400 shim

Table 1. Test results

Test	Shim	V_{ssi} (kN)	V_{ssf} (kN)	V_{ssm} (kN)	V_{ssm}/V_{ssi}	V_{ssm}/V_{ssf}
G300.T1	G300	62.6	99.8	110.3	1.76	1.11
G300.T2	G300	62.3	98.5	113.9	1.83	1.16
G80.T1	G80	66.9	75.0	93	1.39	1.24
G80.T2	G80	51.1	75.8	84.6	1.66	1.12
G400.T1	G400	84.1	107.7	110	1.31	1.02
G400.T2	G400	89.2	103.4	107.4	1.20	1.04

3.2 *Sliding capacity and residual stiffness*

The use of G400 shims changes the bolt sliding capacities from current design values based on the model of the interaction between moment, shear, and normal forces in the bolt during sliding. Details can be found in MacRae et. al (2010). Tests were therefore conducted on M24 bolts with 12 mm to 20 mm thick cleats, and M30 bolts with 20 mm and 25 mm thick cleats to determine these effects. M24 bolts were also tested with Belleville Springs manufactured by Solon Manufacturing, Ohio (Solon Manufacturing 2009). The tests also investigated the residual joint strength, which is defined as the force at which the joint starts to slide and is a key parameter in the SCSHJ development. This was achieved with a series of load regimes, developed as described below.

The specimens were tested with a dynamic load regime to determine their sliding shear capacity (V_{ss}). This was based on the SAC Joint Venture (2000) recommended loading regime for MRSFs. The test rig was unable to accommodate storey drifts, and joint rotations were used instead, but with the non-damaging cycles under 0.005 rad removed. The sequence of the regime was also changed from the stepwise increase to maximum displacement, to a stepwise increase to maximum in half the cycles and back to minimum with the other half. This better reflects the pattern of strong shaking in a severe earthquake, where the peak shaking is reached in the first half to middle of the duration of strong shaking, rather than at the end. This difference is not important when testing for ductility capacity of a specimen but is very important in this case where the residual joint strength is to be determined. If the joint came to rest from the maximum cycle of loading, the position of the bolts would be in the sliding state (Fig 3c), with different residual joint strengths in each direction. Loaded in the same direction, the connection would develop the stable sliding capacity before sliding. In the opposite direction, it is effectively a load reversal and the joint slides at very low loads. This is not representative of actual post-earthquake conditions. The stepwise decrease in cycles ensured the bolts were close to vertical. The test was 3 cycles to 2.3 mm and 3.5 mm, 2 cycles to 4.6 mm, 1 cycle to 6.9 mm and 9.2 mm, 2 cycles to 13.8 mm, and back down again. This corresponds to rotations of 0.005 rad, 0.0075 rad, 0.010 rad, 0.015 rad, 0.020 rad and 0.030 rad.

Following the strong shaking regime described above, a tension load (column rotating away) was applied pseudostatically at a rate of 1 mm/minute at the specimen level. This determined the force required to re-initiate sliding. An intermediate dynamic load regime of 5 cycles each to 3.5 mm and 2.3 mm was then applied to re-center the joint to the original plumb position that again ensured the bolts were close to vertical. A compression pseudostatic load (column rotating in) was then applied to determine the joint strength in the other direction. The pseudostatic tests were then repeated with intermediate dynamic loads in between. The later tests of the same specimens were expected to yield lower elastic strengths due to further degradation from intermediate dynamic and pseudostatic loadings. This was not observed as the additional travel was insufficient to noticeably alter the joint strengths. Later tests were therefore assumed to be representative of the post-earthquake joint conditions.

3.2.1 *Sliding shear capacity*

The measured sliding shear resistance was compared with computed values based on the MacRae et. al (2010) sliding capacity model and shown in Figure 6. The measured values are V_{ssf}, which was used due to better consistency compared to the V_{ssi}. The computed values were modified from the current recommended design values by taking into account the increased shim thickness to 5 mm. As the sliding remained steel on steel, the coefficient of friction was maintained at 0.30. A factor of 1.12 was also applied, which is the ratio of the average tensile strength measured from 2000 tests to the nominal tensile strength by bolt manufacturer Bremick Fasteners (Clifton, 2005).

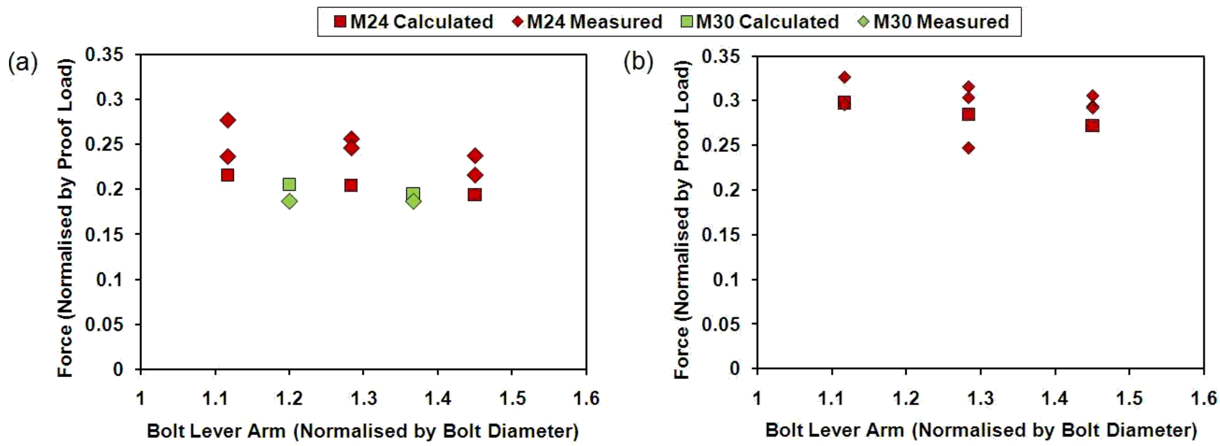

Figure 6: Measured and computed sliding strengths (a) without Belleville Springs and (b) with Belleville Springs

The model matched the measured values reasonably well for bolts with and without Belleville Springs. In both cases for M24 bolts, the V_{ss} increased as the cleat thickness and resulting lever arm decreased. While this trend was not observed in the M30 bolts, this could be due to variations in bolt strengths with only one test of each cleat thickness conducted to date. The model nevertheless underestimated the V_{ss} of the M24 bolts and overestimated the V_{ss} of the M30 bolts. It is speculated that with G400 shims, there is a bolt size factor not considered in the current model that affects the V_{ss}. More data points are required to modify the bolt model for design purposes.

3.2.2 *Post-earthquake stiffness results*

The residual joint strength is largely dependent on the position of the bolts when the joint comes to a complete stop. This was random, and reflected in the reasonably large scatter in results. Figure 7 shows the residual joint strengths as a percentage of the stable sliding shear strength in the dynamic load regime, compared to the pre-earthquake strength. There were considerable differences in the specimens with and without Belleville Springs.

The residual joint strengths without Belleville Springs were generally low and erratic, with averages of 38.1% for M24 bolts, and 39.3% for M30 bolts, compared to the initial strengths of 55.7% and 88.5%. They were similar in both the tension and compression directions. The various sizes performed similarly, apart from the M24 bolts with 12 mm thick cleat. This case had the shortest lever arm of approximately 1.1 (normalised by bolt diameter), and an average residual joint strength of 47.6%. The others had lever arms of 1.2 or higher, and average residual joint strength of 29.4%. These results show that the bolt sliding behaviour is dependent on the lever arm. For thick cleats with long lever arms, moment actions dominate the bolt behaviour which results in more severe inelastic bending and loss in bolt tension. For thinner cleats with short lever arms, the bolt behaviour is dominated by shear actions with less severe inelastic bending and lower loss in bolt tension.

The results with Belleville Springs were more consistent, and had an average of 37.3% for M24 bolts. They also performed better in tension with an average residual joint strength of 40.5%, as opposed to the 34.1% when tested in compression. Unlike the specimens without Belleville Springs, the thicker cleats improved the performance with higher strength. It is acknowledged that these observations were based on limited data, and have yet to be fully understood.

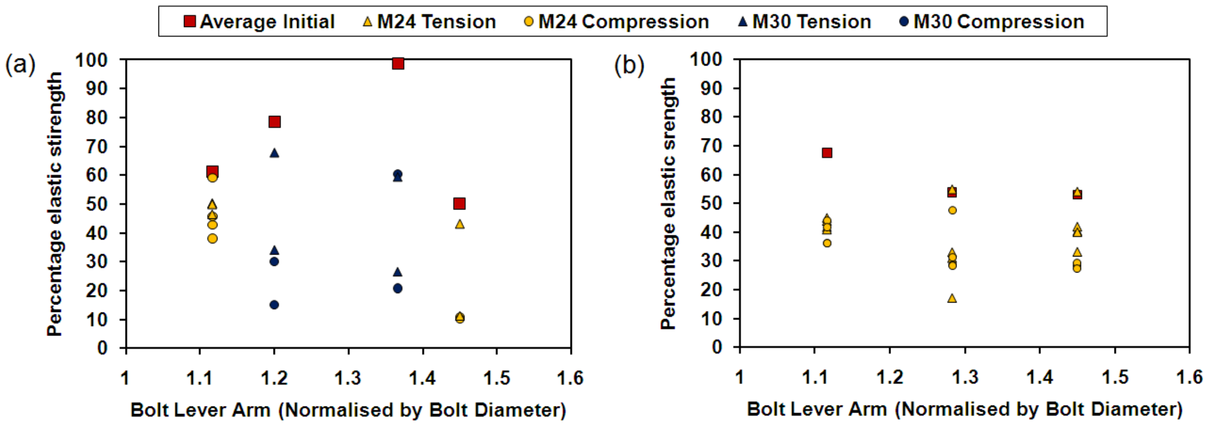

Figure 7: Percentage of retained stiffness in the post-earthquake joint (a) without Belleville Springs and (b) with Belleville Springs

4 FUTURE WORK

Bolt sliding component tests of specimens with M16 to M30 bolts, using different cleat thickness are to be tested. More data and results from different specimen sizes would give a better indication of the effects of bolt size, lever arm length and Belleville Springs on the bolt capacity and residual joint strength. This will then be used to develop a dependable model to determine the expected average residual strengths of the joints. In practice, there will likely be a reasonable amount of variation in the post-earthquake properties of individual joints as indicated by the test results. Nevertheless, the variation and response would even out in a full scale building and the design recommendations will reflect this averaging of results.

Full scale tests on the SCSHJ will be undertaken to determine the joint moment rotational behaviour. This will then be used to modify the SHJ hysteresis curve to undertake further analytical studies of representative frames in RUAUMOKO (Carr 2007). The results will be used to develop design provisions. Cost-benefit analysis will also be conducted to ease the adoption of this system in the industry.

5 CONCLUSIONS

1. The SCSHJ concept described combines bilinear elastic characteristics of the RS with the existing SHJ
2. Abrasion resistant steel with the highest hardness developed the highest and most stable sliding shear capacity, and is recommended for use in future SHJ construction. An example is Bisalloy 400 manufactured by Bisalloy Steels, New South Wales, Australia (Bisalloy Steels 2008).
3. Measured test results were compared to the sliding shear capacity calculated in the sliding capacity bolt model. The bolt capacities and residual joint strengths are affected by bolt size, length of bolt lever arm and Belleville Springs. These effects have not yet been fully understood. At the time of writing, more tests of various specimen sizes are scheduled, which will be used to develop a representative sliding capacity model. The use of sliding capacities calculated from the model will be advocated for future SHJ construction and the post-earthquake strengths used in developing the SCSHJs.

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