Two New Semi-Rigid Joints For Moment-Resisting Steel Frames

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ABSTRACT:

HERA and the University of Auckland are engaged in a long-term research project aimed at developing innovative new semi-rigid joints for moment-resisting steel framed seismic-resisting systems (MRSFs). Two joints are under development, both of which can undergo rotation during a design level severe earthquake and beyond, while suffering minimum structural damage under the design level event.

The first of these joints, termed the *Flange Bolted Joint* (FBJ) is simple to fabricate and erect and is intended for low levels of design ductility demand. The second joint, termed the *Sliding Hinge Joint* (SHJ) is also simple to fabricate but is more complex to erect. It is intended for higher levels of design ductility demand.

Design and detailing provisions for the FBJ are summarised in this paper. The current status of the SHJ in terms of research and design development is also presented, along with the intended future research and design procedure development planned for 2001/2002.

Both these semi-rigid joints offer considerable advantages over traditional rigid jointed MRSFs and have the potential to set the future direction for MRSF seismic-resisting system application in New Zealand.

1 INTRODUCTION AND SCOPE OF PAPER

1.1 *Brief introduction to overall project*

HERA and the University of Auckland are engaged in a long-term research project aimed at developing innovative new forms of semi-rigid joints for moment-resisting steel framed seismic-resisting systems (MRSFs). These joints are intended to remain effectively rigid up to the design level ultimate limit state earthquake moment, eg. as derived from NZS 4203, then to allow rotation to occur between the beam and the column, when this design moment is exceeded. The joint is then designed and detailed to withstand the expected inelastic rotation associated with the design level ultimate limit state earthquake with negligible damage, such that minimum or no repair is necessary when the MRSF has been subjected to that magnitude of earthquake. Finally, the joint is expected to be able to withstand greater levels of inelastic rotation, associated with more severe events, without catastrophic failure, instead undergoing at worst a gradual loss of moment capacity with increasing cyclic rotation demand beyond the design severe seismic level.

Of the five joint types that have been researched to date for this project, two joint details have emerged as preferred options for the beam to column connections of MRSFs. These are *the Flange Bolted Joint* (FBJ) and the *Sliding Hinge Joint* (SHJ).

First results from the overall project were presented in Danner and Clifton (1996). The first draft design provisions for the FBJ and for MRSFs incorporating the FBJ were published in the SESOC Journal, (Clifton et al. 1998). Many progress reports and summary articles on this research have been published between late - 1995 and 2000. A good summary paper, Clifton and Butterworth (2000a), on the status of the project as of late - 1999, was written for the *12th World Conference on Earthquake Engineering*.

Final design and detailing procedures for the FBJ, including a fully worked design example, are presented in the *HERA DCB* Issue No. 58, Clifton and Butterworth (2000b). Design concepts for the SHJ are presented in the *HERA DCB* Issue No. 59, Clifton et al. (2000).

1.2 *Scope of this paper*

This paper presents a brief overview of both semi-rigid joints, covering the following topics:

- Design philosophy and modes of joint behaviour
- Design role of the joint components
- Detailing considerations
- Joint behaviour from experimental testing
- Future developments planned for each joint

Details for the FBJ are presented in section 2, those for the SHJ in section 3. These sections are followed by the acknowledgments and references.

2 THE FLANGE BOLTED JOINT

2.1 *Design philosophy*

The FBJ involves connecting the beam to the column through plates to the beam top and bottom flanges and a plate to the beam web. The pattern of bolts in the flange plates is typical of any flange bolted beam to column connection. The pattern of bolts in the beam web/web plate is unconventional and comprises two horizontal bolt rows, one near the top of the beam web/web plate and the other near the bottom. These are denoted as the *web top bolts* and *web bottom bolts* in Figure 1. Their function is explained below; see especially sections 2.2 and 2.4.

The design philosophy behind this joint has been to establish dependable behavioural characteristics for the FBJ and the MRSF system, for two levels of ultimate limit state earthquake. The first is the design level ultimate limit state earthquake (DLE), as stipulated by NZS 4203, and the second is the more severe maximum considered earthquake (MCE). All the experimental and analytical work undertaken on the FBJ development has been planned and executed with this philosophy in mind.

Under the design level ultimate limit state earthquake, the MRSF with FBJs will respond with minimal structural damage, such that it can be readily repaired.

Under the maximum considered ultimate limit state earthquake, the MRSF with FBJs will retain its integrity, to allow evacuation and post-earthquake assessment, but will suffer controlled joint damage, which will necessitate more extensive repair/replacement of joint components. Even in this case, the joint is designed and constructed so that component replacement can be undertaken (especially the bottom flange plate) without the need to dismantle or to temporarily prop the beam.

2.2 *Design role of joint components*

This section should be read in conjunction with Figure 1. The design role of the joint components is, briefly, as follows:

- All bolts are placed in nominal sized bolt holes, to NZS 3404 Clause 14.3.5.2.1, and are designed to operate in the tension bearing mode, threads included in the shear plane
- The flange bolts and flange plates provide the principal means of developing joint moment capacity, up to an imposed inelastic rotation of twice the design severe seismic rotation limit.
- The web bolts and web plate resist vertical shear and horizontal moment-induced force. Inelastic action is developed in the top and bottom of the web plate under increasing rotation demand. If this demand is sufficient to fracture one or both flange plates, the web bolts and web plate take over the role of moment transfer through the joint.
- The decking support plate, shown as optional in Figure 1, greatly facilitates placement of profiled steel decking around the joint, without the need for temporary propping of the cut edge.
- The top and bottom column stiffeners transfer the flange plate axial forces through the column flange into the web and provide a boundary to the column panel zone.

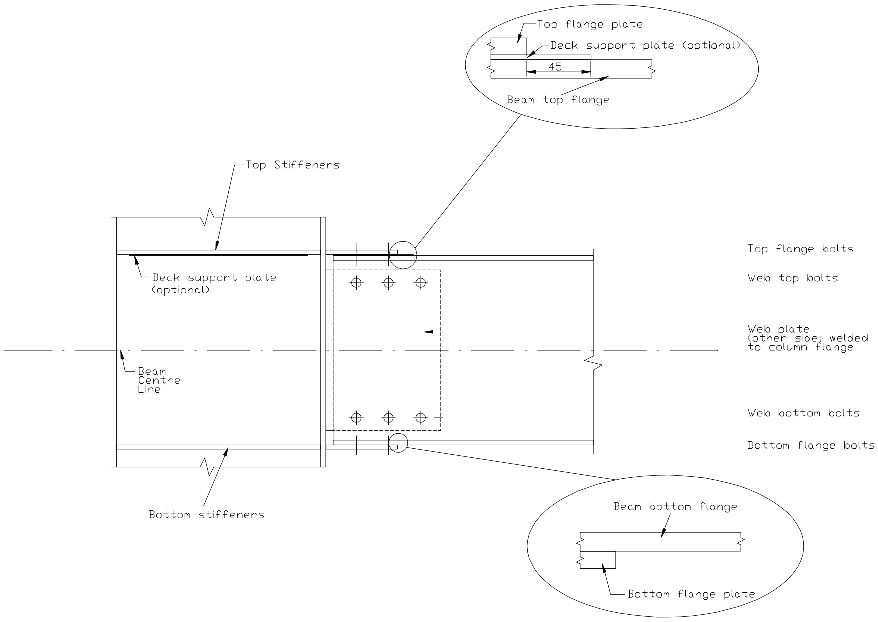


Figure 1 FBJ components and rotation

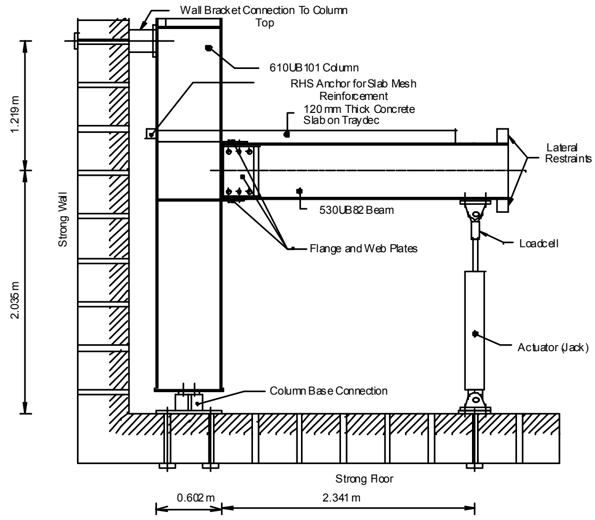
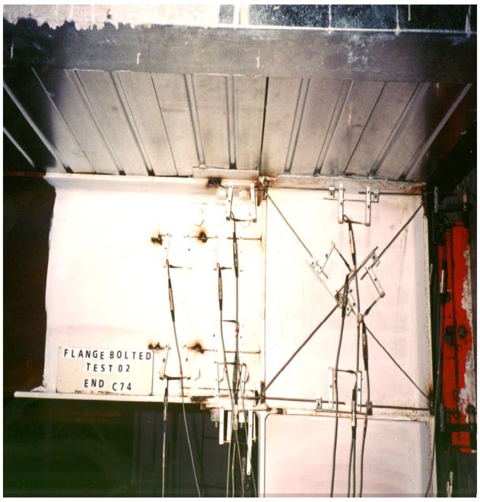


Figure 2 Large-scale test set-up
For the FBJ



to displacement ductility of 6

Figure 3 FBJ Large-scale test 2

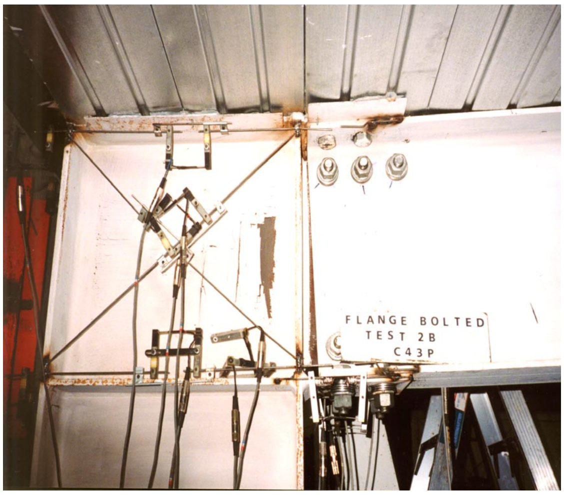
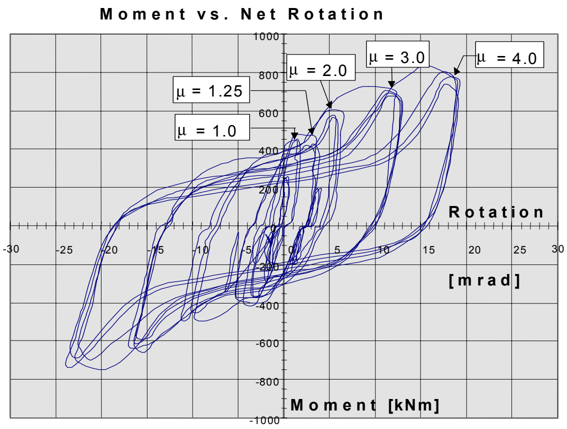


Figure 4 … on third cycle



after testing to displacement of 4

Figure 5 Moment-rotation curves

for testing to twice design ductility demand level

2.3 *Detailing considerations*

These are critical to the joint's performance. They are comprehensively covered in Clifton & Butterworth (2000b) and the key points only are given herein.

Detailing considerations cover:

- Linking of the design displacement ductility for MRSFs using the FBJ to the maximum beam depth used; $\mu_{design} = 2$ for $d_b \leq 930$ mm
- Limit on plate strength (grade) to provide ductility capacity and to suppress bolt shear fracture
- Web plate to be as deep as practicable, to place the web bolts as far apart as practicable
- Edge distances to allow plate inelastic action to develop before bolt tearout
- Clearance between beam face and column flange being sufficient to prevent contact up to at least the design severe seismic rotation demand
- Allowance for manufacturing tolerances
- Constructability
- Surface treatment of ply surfaces where corrosion protection is required

2.4 *FBJ behaviour from experimental tests*

There have been 4 large-scale experimental tests undertaken on the FBJ, involving a 530UB82 beam connected to a 610UB101 column and incorporating a 120mm thick concrete slab on Tray-dec. Fig. 2 shows the experimental test set-up. These 4 tests have comprised two tests on each of two specimens. When developing the FBJ, the detailing of the joint and the strength hierarchy developed within it have been chosen such that:

(i) The joint remains rigid at the serviceability earthquake level, as defined by NZS 4203.

(ii) At the design severe seismic level of rotation demand, bolts can force elongation into the bolt holes and the plates connected to the column, through bearing yielding of the plate/beam elements. This elongation is not to be sufficient to require plate or bolt replacement or significant loss of bolt tension.

(iii) The behaviour of the joint at the design severe seismic level of rotation demand will be maintained up to at least 1.5 times that level of rotation (with increasing yield in the plates but with the bolts retaining their integrity) and repair of the joint at that point will be straightforward to effect.

(iv) At the MCE level of rotation demand, extensive plate/beam element yielding is expected, but bolt fracture does not occur. If the (bottom) flange plate fractures, the horizontal line of web bolts adjacent to the flange provides an alternative horizontal load path for the beam moment-induced axial actions, maintaining a moment capacity similar to that associated with the design joint moment capacity at high rotation demand.

The loading regime for the large-scale tests was tailored to the anticipated modes of operation of the joint as described in (i) – (iv) above, namely:

Part 1 loading regime:

- 3 cycles to nominal serviceability (force controlled)
- 3 cycles to $\mu = 1.0$ (force controlled)
- 3 cycles to $\mu = 1.25$ (displacement controlled)
- 3 cycles to μ_{design} (displacement controlled)
- several rounds of 3 cycles to multiples of μ_{design} (displacement controlled)
- 3 cycles to nominal serviceability (force controlled).

Part 2 loading regime:

At this point, the bolts were slackened off, relubricated and retightened and the joint put through the following:

- 2 cycles to nominal serviceability (force controlled)
- 3 cycles to $\mu = 1.0$ (force controlled)
- 3 cycles to μ_{design} (displacement controlled)
- 3 cycles to $\mu = 6.0$

Figure 3 shows the second test specimen following completion of the part 1 tests. The only visible sign of joint damage following the cycles to $\mu = 4$ (ie $2\mu_{design}$) was hairline cracking in the paint on the bottom flange plate in the immediate vicinity of the bolt holes. (The connection elements were painted to provide visual evidence of yielding and the extent of subsequent inelastic behaviour).

Fig. 5 shows the moment-rotation curve generated by this joint for the part 1 loading. The loading was undertaken up to twice μ_{design} for this beam size (530UB), ie to $\mu = 4$, with excellent results.

Fig. 4 shows the same FBJ, this time from the north side, under peak positive rotation demand on the third cycle of loading to $\mu = 6$ in the part 2 loading.

Observations from these tests showed no visible joint damage on completion of the cycles to $\mu = 2$, first visible indication of bottom flange plate yielding on completion of the cycles to $\mu = 3$ and indications of yielding (but not significant yielding) across the full width of the bottom flange plate at $\mu = 4$.

The part 2 testing (Fig. 4) was intended to determine the experimental behaviour under mode (iv) above. The bottom flange plate fractured on the third cycle to $\mu = 6$ (just before Fig 4 was taken). There was minimal loss of moment capacity at that point, with the web bottom bolts resisting the moment-induced axial tension force and the web bolts as a group resisting the vertical shear force. Continued cycles to $\mu = 6$ resulted in fracturing of the top flange plate and the growth of cracks from the top and bottom of the web plate in towards the centre. The joint continued to develop its calculated design moment at peak rotation until the fourth cycle of loading to $\mu = 6$ and further strength drop-off was gradual and controlled.

The critical component in the FBJ, up to at least 1.5 x the DLE rotation demand, is the connection at the beam flanges. As the large-scale experimental tests could investigate only one bolt size and layout and only at a pseudo-static rate of loading, a series of small-scale tests were undertaken during 1999/2000 on representative flange plate - beam flange connections to determine the influence of the following parameters :

- bolt size - range from M24 to M30
- ratio of design shear capacity of bolt group to plate strength
- ratio of plate thickness to bolt diameter
- effect of repeated loading event on bolts and plate, including after a delay time of 4 weeks
- effect of loading rate; seismic-dynamic and pseudo-static.

Results from these small-scale component tests are referenced from Clifton and Butterworth (2000b).

2.5 *Future developments for the FBJ*

The completion of the experimental testing programme and analytical modelling programme (which is not covered herein - see (Clifton et.al. 1998) for details) allowed the design and detailing provisions to be finalised. These are presented in sections 3 and 4 of Clifton and Butterworth (2000b), with a fully worked design example in section 5.

A spreadsheet programme is available free-of-charge on a “use at your own risk” basis; contact Charles Clifton at : structural@hera.org.nz for a copy. The scope of the program is outlined in Clifton & Butterworth (2000b).

It is planned to compile the results of the FBJ research and design, detailing provisions into a HERA Report during 2001 with connection design tables produced for a range of beam sizes.

3 THE SLIDING HINGE JOINT

3.1 *Design philosophy*

The SHJ involves pinning the beam relative to the column at the beam top flange level, via the top flange bolts and top flange plate (see Fig. 6). This keeps lateral movement under negative-moment-induced tension at the base of the slab to 3 mm, thus minimising undesirable floor slab participation and slab damage. Joint rotation is achieved through controlled sliding at the bottom flange and web bottom bolt level.

The sliding detail comprises the layers shown in Figure 6. The sliding layers are between the brass shims and plate (web plate, bottom row or bottom flange plate). The holes for the web bottom bolts in the web plate and for the bottom flange bolts in the bottom flange plate are slotted to allow this sliding to occur. The beam flange or web and the associated cap plates have nominal sized holes.

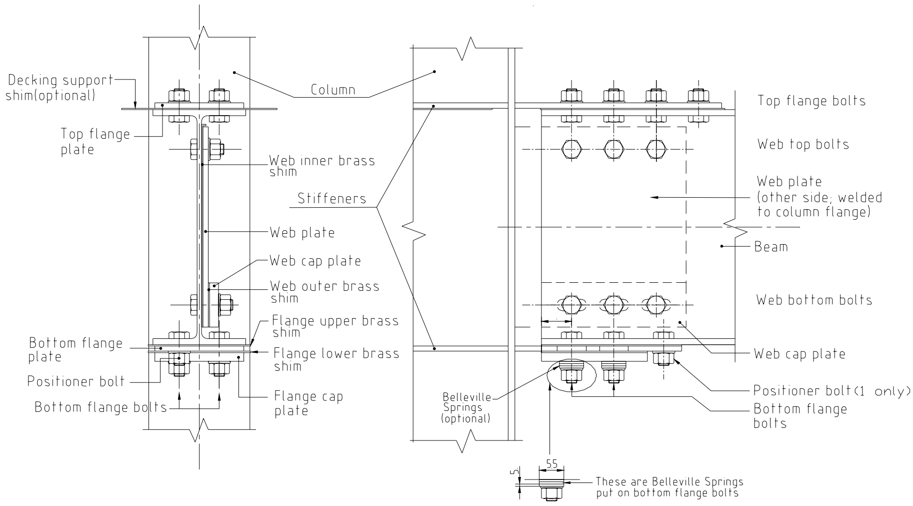


Figure 6 SHJ components and rotation

inelastic negative rotation

Figure 7 Large-scale SHJ
test 3 showing set-up
of specimen and floor slab

Figure 8 SHJ test 3:
cycle to design inelastic negative rotation

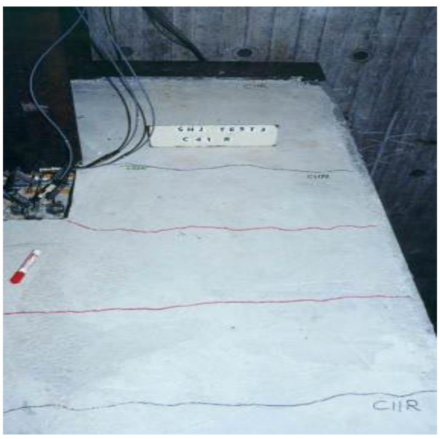


Figure 9 SHJ test 3:
Condition of concrete slab at design

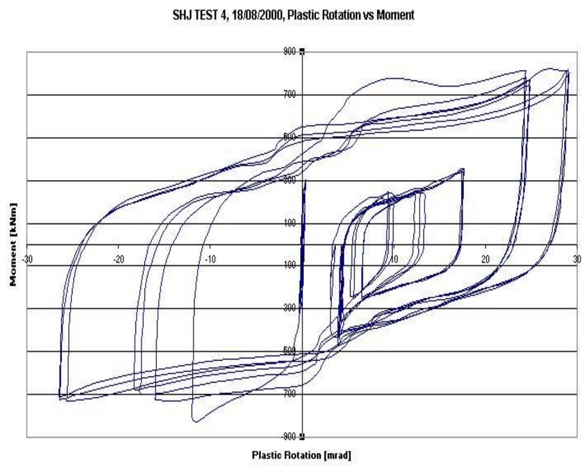


Figure 10 SHJ test 4: inelastic

rotation versus moment

When the moment demand on the SHJ from earthquake generates internal beam axial forces which exceed the sliding resistance available through the bottom flange bolts and web bottom bolts, the joint will slide, allowing beam rotation to occur. As sliding occurs, the cap plate is anchored in position relative to the beam flange or web by the bolts, allowing the cap plate to slide relative to these surfaces. Once the imposed moment reduces, there comes a point where the sliding stops and the joint becomes effectively rigid again. This is illustrated in Figure 10, which shows the joint plastic rotation versus moment from one of the SHJ Large-Scale tests.

The slotted holes at bottom flange and web level are designed to accommodate a joint rotation of ± 30 mrad; if the inelastic rotation demand exceeds this, the joint undergoes further inelastic behaviour through flange plate yielding, in the same manner as for the FBJ (see section 2.2). The first large-scale SHJ specimen, tested to destruction in test 2, still developed its design moment capacity at over 120 mrad rotation.

3.2 *Design role of joint components*

This section very briefly outlines the design role of each of the joint components. For more detail, see (Clifton et al. 2000). It should be read in conjunction with Figure 6:

- The top flange bolts act as the anchor point for joint rotation, pinning the beam top corner in place relative to the column.
- The web bottom bolts and bottom flange bolts develop the sliding shear resistance
- The joint design moment capacity is developed by taking moments about the top of steel beam of the web bottom bolts and bottom flange bolts design sliding shear capacity
- The web top bolts resist the applied vertical shear force. They are subject to only small lateral movement due to their proximity to the top flange bolts.
- The cap plates provide the support to the end of the sliding bolt groups remote from the beam
- The brass shims facilitate smooth sliding between the steel surfaces at a near constant level of shear friction, which is essential to the maintenance of stable bolt tension and hence constant sliding shear resistance when the joint is sliding
- The Belleville Springs, which are optional additions to the bottom flange bolts, assist these bolts to retain bolt tension under sliding. This increases the retention of bolt sliding shear capacity and retains joint stiffness in the post-sliding regime of behaviour
- The positioner bolt is a black finish class 4.6 bolt that connects between the beam flange and bottom flange plate only, through nominal sized holes in each ply. It has the same diameter as the rest of the bolts (which are all galvanised finish property class 8.8 structural bolts). The positioner bolt has three very important roles, namely:
  (1) It acts as a stability bolt for erection purposes, making the joint rigid for erection by developing moment resistance in conjunction with the top flange bolts
  (2) It functions as a locater bolt for the sliding bolts, ensuring that they are located in the middle of the slotted holes in the erected joint
  (3) It provides a rapid visual indicator as to whether the joint has gone into the sliding mode following a severe earthquake; if this happens and the joint inelastic rotation exceeds around 10 mrad, the positioner bolt shears through and the lower half drops out.

As with the FBJ, detailing considerations are important to making the SHJ's performance satisfactory. There is insufficient space herein to present these; refer to (Clifton et al. 2000) for this information.

3.3 *SHJ behaviour from experimental tests*

There have been 4 large-scale experimental tests undertaken on the SHJ, involving the same beam/column/slab setup as for the FBJ (see Fig. 2) and a bolt layout as shown in figure 6.

Performance of the first test specimen (tests 1 & 2) was unsatisfactory in a couple of regards; amendments to the design and detailing procedure were made, with a second test specimen built and tested in tests 3 and 4. Performance of this specimen was good.

Figure 7 shows the test 3 prior to commencing loading. Figure 8 shows the joint under design inelastic negative rotation, ie. to an actual joint inelastic rotation of 25 mrad.

At that level of negative rotation in a conventional joint, the slab would have developed severe cracking over the joint at the face of the column, with fracture of the slab mesh reinforcement. With this joint, only minor slab cracking and no mesh reinforcement fracture had developed at that level of rotation, as shown in Figure 9.

Once the sliding surfaces commence sliding, these surfaces move with little increase in force, up until near the end of the slotted hole is reached, as shown in Figure 10. Contrast this with the considerable increase in moment developed by the SBJ at lower levels of inelastic rotation, as shown in Figure 5.

3.4 *Future developments for the SHJ*

The final aspects of design and detailing for the SHJ have still to be determined, however the principal provisions are completed and are presented in (Clifton et al. 2000).

The moment-rotation characteristics of the joint are, not unexpectedly, significantly different from those of other bolted joints, as shown in Figure 10. The presence of Belleville Springs (see Fig. 6) on the bottom flange bolts also changes these characteristics. Mathematical models of these characteristics have been developed for input into the numerical integration time history program RUAUMOKO, Carr (2000)

Over 2001 the following will be undertaken:

- The design procedure will be put onto spreadsheet
- Representative 5 and 10 storey MRSFs incorporating the SHJ will be designed
- Numerical integration time-history analyses on these frames will be undertaken to determine the system response and ductility demand on the joints
- The effect of potential joint misalignment on the moment-rotation characteristics will be investigated

4 ACKNOWLEDGEMENTS

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(1) The undergraduate students from Germany who have undertaken and continue to undertake the setting up of experimental tests, the processing and presentation of data from this testing, the development of analytical modelling data and other essential work.

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(3) John Butterworth, for his guidance and input as principal PhD supervisor to Charles Clifton.

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