

MOMENT-RESISTING STEEL FRAMED SEISMIC-RESISTING SYSTEMS WITH SEMI-RIGID CONNECTIONS

Charles G CLIFTON¹ And John W BUTTERWORTH²

SUMMARY

This paper presents an overview, as of August 1999, of the results of a long-term research project aimed at developing an innovative form of moment-resisting steel framed seismic-resisting system (MRSF) that can be readily brought back into service after a severe earthquake. The system utilises standard I-section members for the beams and columns, with these members connected via semi-rigid joints that can rotate (open and close) during a design level severe earthquake and beyond, while suffering minimum structural damage under the design level event.

This project commenced in 1994 and three different forms of semi-rigid joint have been considered. Of these, two have been shown to be not viable, for reasons of cost, constructability and performance. However, one form of joint, the flange bolted joint (FBJ), is showing considerable promise and three variations of this joint are now actively being researched.

This paper first presents an introduction to the overall project. It then presents the design philosophy and target performance requirements that have been formulated for these joints, followed by the FBJ modes of behaviour and the experimental and analytical studies underway on each type of joint. It presents the status of research, up to August 1999, and the intended future research and design guidance development into 2000.

INTRODUCTION TO THE OVERALL PROJECT

This is a long-term research project. Its outcome will be the development of an innovative form of MRSF seismic-resisting system that can readily be brought back into service after a severe earthquake, by tailoring the ductility demand on the joints with their ductility capability at target levels of severe seismic demand.

The overall project commenced in 1994/95 and involves the following sequence of operations:

- (1) Establishment of a design philosophy and set of target performance requirements to be met for the connections and the overall structural system under severe seismic conditions.
- (2) Development of potentially suitable connection details between the beams and columns and at the column base. A number of connection types have been considered and the most suitable of these are currently being researched.

For a given type of joint, this development process has involved:

- (2.1) Ascertaining the likely modes of joint behaviour and performance under seismic conditions.
- (2.2) Design of representative examples of the joint for experimental testing to determine the actual performance, the moment-rotation characteristics and the ductility capacity.
- (2.3) Development of analytical models of the joint's moment-rotation characteristics for use in analytical modelling of MRSF systems incorporating this joint.

¹ Structural Engineer, HERA

² Department of Civil and Resource Engineering, University of Auckland

- (2.4) Undertaking time-history analyses of representative systems to determine rotation demand on the joints and hence to confirm that the expected rotation demand is within the ductility capability of the joint.
- (2.5) Considering constructability and cost issues in order to show that the given joint is a viable option for a MRSF system.

This process is ongoing and iterative for each joint considered, involving several cycles of design/experimental testing/analysis/evaluation/design modification, etc.

For joints which are shown to be viable after going through this process;

- (3) Development of design procedures and detailing requirements for the joint and overall MRSF system.

Over the period from 1994/95 to 1999, three types of joint have been considered.

The first joint type considered comprises the beams connected to the columns with a bolted flush endplate connection, with compressible ring-spring assemblages under the bolts. See Figure 1; a detailed joint description is given in [Danner and Clifton, 1995]. (This is termed a *ring-spring* joint).

The ring-spring joint has been shown to be viable as a connection between beams and columns in terms of performance, but is difficult to construct and unrealistically expensive. It is, however, suited to application at column bases. A preliminary design procedure for its use in that application has been developed [Clifton et.al., 1998a], however detailing aspects still need more consideration.

The second joint type initially considered (over the period 1995 to 1997) involved the beams being post-tensioned to the columns with a pair of post-tensioning bars placed about the beam centreline (termed a *tendon joint*). See details in Figure 2; a detailed joint description is given in [Danner & Clifton, 1995]. This joint type has subsequently been shown to be not viable in a MRSF for performance and constructability reasons, details of which are given in [Clifton et.al., 1998b].

As of August 1997, experimental tests on the ring-spring joint and the joint issues requiring design consideration for the tendon joint had indicated that, while the semi-rigid joint concept is feasible, both these joint systems have shortcomings, in terms of cost and constructability. These problems are especially acute in the case of the tendon joint.

A detailed evaluation of potential joint options for ongoing consideration was undertaken from October 1997 to January 1998, giving rise to the concept of the *flange bolted joint (FBJ)*. Three variations of FBJ were considered, being the *standard FBJ*, as shown in Figure 3, the *FBJ with brass shims*, as shown in Figure 4 and the FBJ with sliding bottom flange, as shown in Figures 5 and 6.

The first two types of FBJ (standard and brass shims) are viable from a cost and constructability point of view and experimental testing on each commenced with large-scale, pseudo-static tests in 1998. These tests showed the joints' performance as acceptable, under pseudo-static conditions. Smaller scale tests on critical joint components are currently underway under both pseudo and seismic-dynamic loading conditions. Initial results confirm the viability of the standard FBJ, which is designed for low ductility demand, using $\mu = 2$ to NZS 4203 [NZS 4203, 1992] and NZS 3404 [NZS 3404, 1997].

The FBJ with sliding bottom flange is intended for $\mu = 4$ applications. Currently (August 1999) a tentative design procedure has been developed and experimental testing of the bottom flange sub-assembly (see Figure 6) at both pseudo-static and seismic-dynamic rates of loading is about to commence. Full scale, pseudo-static tests (similar in size to those shown in Figures 3 and 4) are planned for early 2000.

DESIGN PHILOSOPHY AND TARGET PERFORMANCE REQUIREMENTS

Design Philosophy

The design philosophy being developed aims to establish dependable behavioural characteristics for the semi-rigid MRSF seismic-resisting system for two levels of ultimate limit state seismic event. The first is the design level earthquake, as specified by NZS 4203, and the second is a defined maximum credible earthquake – ie. a

more severe event than the ultimate limit state design earthquake. Details of each level are given in section 6.4 of [Clifton et.al., 1996]

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

Under the maximum credible ultimate limit state earthquake, the MRSF is expected to retain its integrity, to allow evacuation and post-earthquake assessment, but to suffer controlled structural damage.

In terms of the current force-based seismic design philosophy, it is intended that the design procedures developed for these semi-rigid systems utilise the equivalent static method of NZS 4203 Clause 4.8, in conjunction with NZS 3404 and, where appropriate, HERA Report R4-76 [Feeney and Clifton, 1995]. This is to ensure maximum ease of use. The preliminary sizing/design method, in particular, must be easy and rapid to use.

Target Performance Requirements

The target performance behavioural requirements from each system for the two levels of earthquake described in section 2.1 are as follows:

- (1) For the design ultimate limit state earthquake (ie. as represented by NZS 4203 Section 4, involving at least a 450 year return period);
 - (i) Negligible inelastic demand in the beams
 - (ii) Minimal inelastic demand in the columns at base level (such that the column bases will be readily repairable) and none at higher levels
 - (iii) The rotation demand on the joints not to exceed that associated with easy assessment and, where necessary, rapid and straight-forward repair
 - (iv) Column panel zones to remain essentially elastic
 - (v) Lateral drift not to exceed 2%
 - (vi) Lateral stiffness at the end of the elastic range of behaviour to be sufficiently great to minimise P- Δ effects

- (2) For the maximum credible earthquake (ie. based on at least a 1000 year return period);
 - (i) Negligible inelastic demand in the beams, except in the vicinity of bolts to beam flange and web elements.
 - (ii) Inelastic demand in the columns to be able to be dependably resisted (this applies especially at the base)
 - (iii) Joint rotation demand may cause significant local element damage, but no joint failure. Repair still to be possible.
 - (iv) Panel zones may yield to accommodate increased joint moments from (iii) above.
 - (v) Lateral drift to be within sustainable limits, including the influence of P- Δ effects.

Application of the design procedures for the force-based method of design, in practice, will involve

- (a) Analysing the frame for the design level earthquake using, typically, the Equivalent Static Method from NZS 4203, and sizing the members and connection components to meet the required strength and stiffness criteria for this event.
- (b) Developing a suitable strength hierarchy within the connection components and detailing the system such that the performance conditions specified in (2) above can be met for the maximum credible earthquake.

One of the principal long-term aims of this research project is to determine the seismic behaviour of each system, in order to develop simple design and detailing guidelines for designers to apply, in accordance with typical design office practice, so as to meet the performance requirements of (1) and (2) above. These design and detailing guidelines will avoid designers needing to explicitly demonstrate compliance with the above performance requirements.

FLANGE BOLTED JOINT DEVELOPMENT

Joint Details and Modes of Operation

Three flange bolted joint (FBJ) details are being developed, termed the standard FBJ, the FBJ with brass shims and the FBJ with sliding bottom flange.

The standard joint is intended for application in nominally ductile systems (category 3 but using $\mu = 2.0$ to NZS 3404, the brass shimmed joint in limited ductile (category 2) systems and the sliding bottom flange joint in fully ductile (category 1) systems, probably with $\mu = 4.0$.

Figure 3 shows the standard FBJ tested in July/August 1998. Figure 4 shows the FBJ with brass shims tested in April 1998. The only differences between the joints are that the latter incorporate brass shims and Belleville springs. (The Belleville springs are truncated conical spring washers, which are elastically compressed during bolt tensioning and help to maintain bolt tension during imposed inelastic rotation).

The first two flange bolted details utilise nominal sized holes, to NZS 3404 Clause 14.3.5.2.1. The bolts are fully tensioned into these holes by the part-turn method specified in NZS 3404. The detailing of the flange and web plates, edge distances, ratio of bolt diameter to plate thickness are all 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 and repair of the joint at that point will be straight-forward to effect.
- (iv) At the maximum credible 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 reasonable moment capacity at high rotation demand.

The third flange bolted joint detail, ie. that shown in Figures 5 and 6, is intended to function somewhat differently. The beam is pinned laterally at the top flange and the top line of web bolts, using a standard FBJ detail. This keeps lateral movement in the floor slab to $\pm 2-3\text{mm}$, thus minimising undesirable floor slab participation and slab damage. Joint rotation is achieved through sliding at the bottom flange and web level. The bolts operate in a friction sliding mode along two interfaces, one of which is between the cap plate and cleat. Friction sliding along this interface is achieved by the bolts dragging the cap plate along in line with the beam flange.

As of mid-August 1999, the anticipated modes of behaviour of the FBJ with sliding bottom flange have been developed and potential design capacity determined. This involves:

- (a) The joint remaining rigid at the serviceability earthquake level.
- (b) The bottom flange sliding within the slotted holes, up to a level of rotation demand equal to the design severe seismic level of rotation demand multiplied by an over-rotation factor (say 1.25).
- (c) At rotation demands in excess of (b), the flange plate will undergo yielding, but the bolts will not lose significant tension. Thus the sliding stiffness of the joint will be maintained.
- (d) If subjected to rotation demand well in excess of (c), the joint will sustain a significant moment capacity by the same mechanism described in (iv) above for the other two FBJ's.

Experimental Testing

A full-size flange bolted joint with brass shims was tested in April 1998. The test assemblage comprised a 530UB82 beam supporting a 120mm thick floor slab on Traydec and connected to a 610UB101 column. See details in Figure 4.

A full-size test on a standard flange bolted joint with the same beam, column and slab details (see Figure 3) was undertaken in July/August 1998.

The loading regime for both these tests was tailored to the anticipated modes of operation as described in (i) - (iv) of section 3.1, 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)
- 3 cycles to $1.5 \times \mu_{\text{design}}$ (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 $\mu = 6.0$.

Fig. 7 shows the moment-rotation curve from the part 1 test on the standard flange bolted joint.

Both joints exhibited excellent performance, meeting the criteria expressed in section 4.1. Some results of these tests are published in [Clifton et.al., 1998a]; more details will be in [Clifton et.al., 1999].

As already mentioned in section 1 herein, small scale component testing on the beam flange to flange plate components of all three joint types will be undertaken over August 1999 to January 2000. Results relating to the standard FBJ and the FBJ with brass shims, along with final design procedures for these joints, will be published in [Clifton et.al., 1999].

A full scale test on the FBJ with sliding bottom flange is planned for early February, 2000.

Analytical Modelling

As described in items (2.1) to (2.4) of section 1, the analytical modelling is being undertaken in order to determine how well the proposed frame/joint systems are expected to meet the target performance requirements specified in section 2.

The analytical modelling has been undertaken on the perimeter frames for five and ten storey, rectangular in plan buildings of 35 x 21 metre footprint. They carry loads typical [Clifton et.al., 1996] of an office building. Each building is supported laterally by a perimeter MRSF along each wall. Fig. 8 shows an elevation of one such frame, illustrating the features incorporated into the analytical model.

As of August 1999, analyses on a range of five and ten storey buildings incorporating standard FBJs and FBJs with brass shims, situated in low and high seismic zones to NZS 4203, have been undertaken. Fig. 9 shows an output from one analysis. Results from the five storey analyses are published in [Clifton et.al., 1998a]; further results will be in [Clifton et.al., 1999]. These results show the system response for the first two FBJs will generally meet the performance expectations expressed in section 2.

Design Procedure Development

Initial design procedures for the standard FBJ and the FBJ with brass shims were presented in [Clifton et.al., 1998a], with updates in [Clifton, 1998]. Results from the initial small-scale component tests have shown that modifications to the design procedure in [Clifton et.al., 1998a; Clifton, 1998] are required; these will be presented in [Clifton et.al., 1999].

A preliminary design procedure for the FBJ with sliding bottom flange will be presented in [Clifton et.al., 1999] and further reviewed in light of the large-scale experimental tests planned for February 2000. It is planned to publish a final design procedure for this joint towards the end of 2000.

CONCLUSIONS

MRSFs with semi-rigid joints can offer advantages over traditional, rigid jointed MRSF seismic-resisting systems. HERA is involved in a long-term research programme aimed at developing suitable joints for this application. The concept of the Flange Bolted Joint (FBJ) shows considerable promise. This paper has introduced the three FBJ types currently being researched, the status of that research (up to August 1999) and the intended future research into 2000.

ACKNOWLEDGEMENTS

The principal author, Charles Clifton, would like to acknowledge the contribution of all persons/organisations involved in this research, with special mention of:

- (1) The undergraduate students from Germany who have worked on this project during their study leave at HERA, especially the immediate past students, Sascha Louis, Rüdiger Nübling and Martin Eisenbeis and current students Chris Maurer and Frank Haunstetter.
- (2) Dr John Butterworth, Dr Jos Geurts and Hank Mooy, University of Auckland, for assistance with planning and undertaking the experimental testing.
- (3) Dr John Butterworth, for his guidance and input as principal PhD supervisor to Charles Clifton.
- (4) The Foundation for Research, Science and Technology, for providing the principal past and ongoing funding for this project, through Contracts HER602 and HER801.

REFERENCES

- Clifton, G.C. et.al. (1996), *Development of Moment-Resisting Steel Frames Incorporating Semi-Rigid Elastic Joints: 1995/96 Research Report*, HERA Report R4-88, New Zealand HERA.
- Clifton, G.C. (1998), "Moment-resisting steel framed seismic-resisting systems with semi-rigid connections: update on research, design requirements", *HERA DCB*, Issue No. 47, pp. 12-16.
- Clifton, G.C. et.al. (1998a) , "Moment-resisting steel framed seismic-resisting systems with semi-rigid connections", *SESOC Journal*, Vol. 11, No. 2, pp 21-41 and 43-52.
- Clifton, G.C. et.al. (1998b), *Development of Perimeter Moment-Resisting Steel Frames Incorporating Semi-Rigid Elastic Joints: 1996/97 Research Report*, HERA Report R4-94, New Zealand HERA.
- Clifton, G.C. et.al. (1999), *Development of Moment-Resisting Steel Frames Incorporating Semi-Rigid Joints: 1997/99 Research Report*, HERA Report R4-98, New Zealand HERA, (to be published).
- Danner, M. and Clifton, G.C. (1995), *Development of Moment-Resisting Steel Frames incorporating Semi-Rigid Elastic Joints: 1994/95 Research Report*, HERA Report R4-87, New Zealand HERA.
- Feeney, M.J. and Clifton, G.C. (1995), *Seismic Design Procedures for Steel Structures*, HERA Report R4-76, New Zealand HERA.
- NZS 3404 (1997), *Steel Structures Standard*; Standards New Zealand.
- NZS 4203 (1992), *Loadings Standard*, Standards New Zealand.

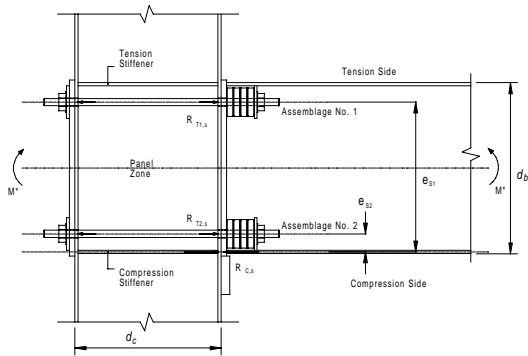


Figure 1 Ring-spring joint in closed condition

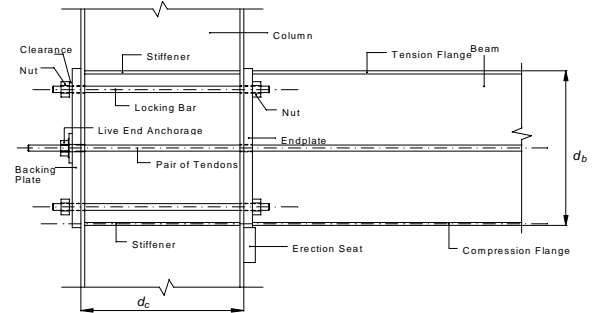


Figure 2 Post-tensioned tendon joint in unloaded condition

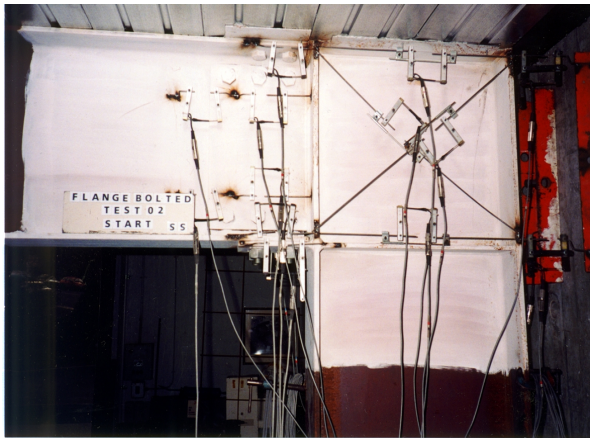


Figure 3 Standard flange bolted joint at start of test

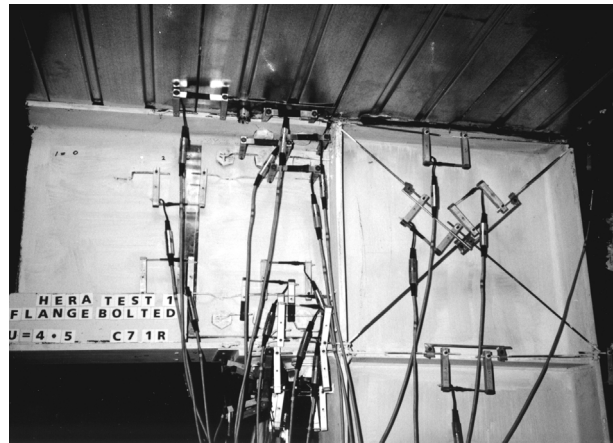


Figure 4 Flange bolted joint with brass shims under test

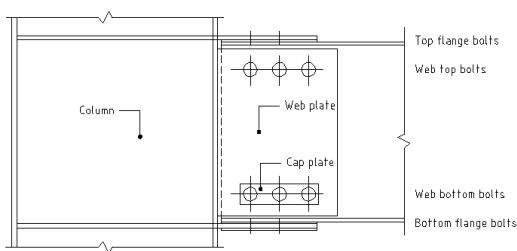


Figure 5 Simplified proposed flange bolted joint with sliding bottom flange

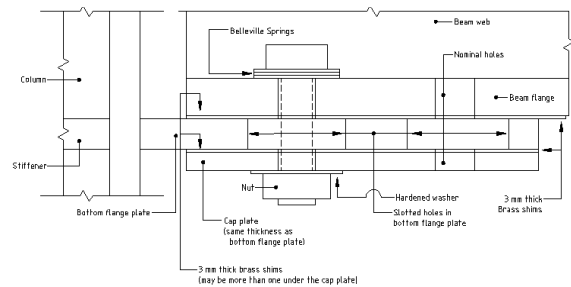


Figure 6 Enlarged view of proposed bottom flange detail for FBJ with sliding bottom flange

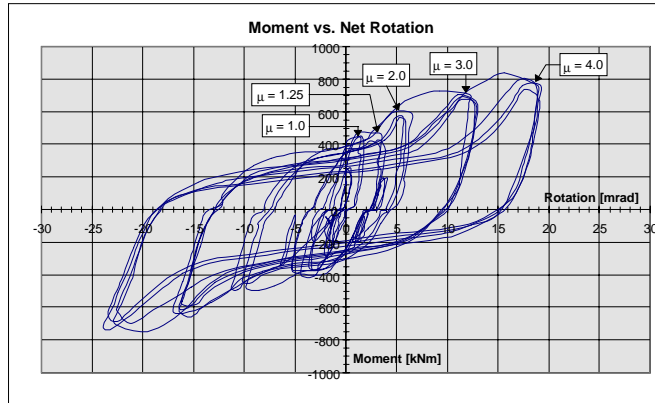


Figure 7 Moment versus net rotation curve for the standard flange bolted joint

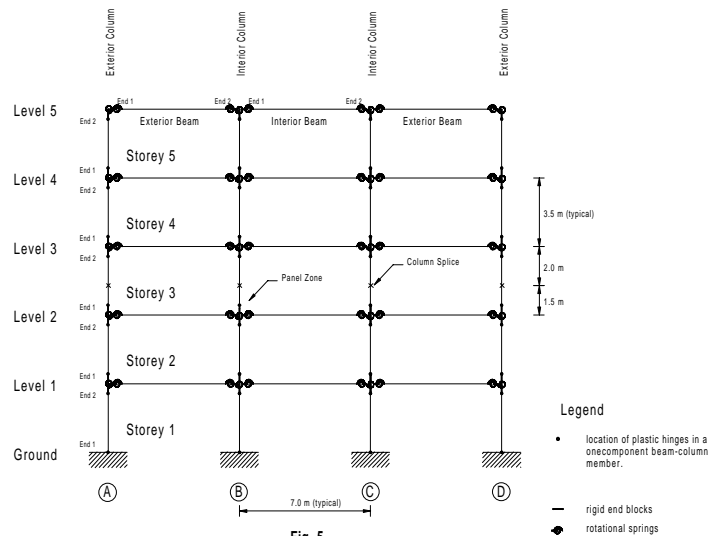


Figure 8 Elevation of 5 storey frame model for analytical studies

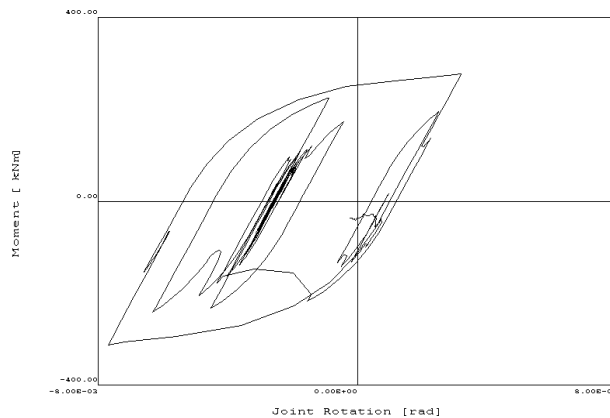


Figure 9 Moment-rotation curve from end 1, exterior beam, level 5, for Wellington frame, standard FBJ, under 1.66 times Ell Centro 1940 North-South record