Experimental Investigation of Cold-formed Steel Beam-column Sub-frames: Pilot Study

M. F. Wong

K. F. Chung

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EXPERIMENTAL INVESTIGATION OF COLD-FORMED STEEL BEAM-COLUMN SUB-FRAMES: PILOT STUDY

M F Wong[1] and K F Chung[2]

SUMMARY
This paper presents the findings of an experimental investigation on the structural performance of bolted moment connections in cold-formed steel beam-column sub-frames. A total of eight tests with three different connection configurations in both internal and external columns were carried out. Double lipped C-sections back-to-back with hot rolled steel gusset plates of 10 mm and of 16 mm in two different shapes were tested; four bolts per member were used in the connections.

Among the tests, three different modes of failure were identified and the measured moment resistances at the connections were found to vary from 36% to 97% of the measured moment capacities of the cold-formed steel sections, demonstrating that bolted moment connections between cold-formed steel members are structurally feasible and economical. Furthermore, structural members with double lipped C sections back-to-back are shown to be practical in constructing short to medium span portal frames with bolted moment connections through rational design.

INTRODUCTION
Cold-formed steel sections are light-weight materials and suitable for low-rise building construction owing to its high buildability[1]. The most common section is lipped C section, and the thickness typically ranges from 1.2 mm to 3.2 mm. The yield strengths normally are 280 N/mm^2 to 450 N/mm^2. Cold-formed steel sections are widely used as purlin members in modern roof systems, floor joists of medium span, studs in wall panels, storage racking, and hoarding structures.

Both bolts and self-drilling self tapping screws are common fasteners in cold-formed steel construction. While many modern codes and standards[2,3,4,5] for cold-formed steel structures present design expressions on the load carrying capacities of bolts and screws, there is little design recommendations on the strength and the stiffness of connections. The minimum configuration of bolted connections with two bolts per member is commonly regarded as simple (or shear) connections.

[1]Research student, and [2]Associate Professor, Department of Civil and Structural Engineering, the Hong Kong Polytechnic University, Hong Kong, China.

Bolted moment connections with high strength and stiffness are essential in safe and economical design and construction of purlin systems[6,7], and thus there are many research works reported in the literature on the development of purlin-rafter connections in modern roof systems. They are basically beam-to-beam connections with different degrees of continuity to reduce mid-span moment and deflection. However, most of the modern codes and standards do not consider connections between cold-formed steel sections to be moment resisting, and thus many new cold-formed steel products are developed from experimental testing rather than from design methods due to the lack of relevant design recommendations.

In order to extend the effective use of cold-formed steel in building applications, it is highly desirable to build safe and economical moment frames, and thus design information on bolted moment connections in beam-column sub-frames with practical connection configurations should be provided. Several investigations on cold-formed steel members with bolted moment connections have been reported in the literature[8,9,10].

This paper presents the findings of a test program on the investigation of the structural performance of bolted moment connections in cold-formed steel beam-column sub-frames. A total of eight tests with three different connection configurations in both internal and external beam-column sub-frames were carried out. It is intended to demonstrate the high structural performance of bolted moment connections between cold-formed steel members, and to establish the high structural efficiency of bolted moment connections in practical framing.

TEST PROGRAM

The basic configuration of bolted moment connections proposed in beam-column sub-frames for building applications is:

- All structural members such as beams and columns are formed with two lipped C sections back-to-back with interconnections.
- Moment connections between beams and columns are formed with hot-rolled steel gusset plates.
- Only the webs of lipped C sections are bolted onto gusset plates.
- Four bolts per member are used.

The connection details are rationalized after considering ease of fabrication and installation. In general, the proposed moment connections are not able to develop full moment capacity of the connected members due to discontinuity of load paths along section flanges in the sections. In the eight beam-column sub-frame tests, two lipped C sections with different thickness are used:

- C15016 DS denotes a double section of lipped C sections of 150 mm section depth, 64 mm flange width with a thickness of 1.6 mm.
- C15020 DS denotes a double section of lipped C sections of 150 mm section depth, 60 mm flange width with a thickness of 2.0 mm.

The design yield strength of all the sections is 450 N/mm^2 which is designated as G450. The moment capacities of the sections C15016DS G450 and C15020DS G450 measured from four point load tests are 16.95 kNm and 21.36 kNm respectively. All bolts are 16 mm in diameter and of Grade 8.8.

All the test specimens are constructed according to the basic configurations with systematic variations in the connection details, i.e. bolt arrangement (or bolt pitch), and shape and thickness of gusset plates. The test designation 'S090A1' refers to an internal beam-column sub-frame with 'cross' shaped gusset plate of 10 mm thick and 4 bolts per member at a bolt pitch of 90 mm. The test program is summarized in Table 1.

In order to examine the structural performance of bolted moment connections against bolt pitches, three internal beam-column sub-frame tests with different pitches of 90 mm, 180 mm and 240 mm were carried out, i.e. S090A1, S180A1 and S240A1; the thickness of the gusset plates is 10 mm. Two external beam-column sub-frame tests with similar connection configurations were also carried out for comparison, i.e. E180C1 and E240C1.

Furthermore, in order to examine the structural performance of bolted moment connections against gusset plate thickness, two internal beam-column sub-frame tests with gusset plates of 10 mm and 16 mm thick were carried out, i.e. S090A2 and S180A2. One additional external beam-column sub-frame with a gusset plate of 16 mm thick was also carried out for comparison, i.e. E180C2.

TEST INSTRUMENTATION

The general arrangement of the test set-up together with the proposed connection configurations for both internal and external beam-column sub-frames are shown in Figure 1. Both the applied load and the displacements of each member of the test specimens were measured during the entire deformation history. The tests were terminated when either section failure or member buckling occurred, or deformation of the test specimens became excessive, i.e. over 250 mm. In most cases, a pre-load of 2 kN was applied before tests to ensure that all bolts were in contact with the section webs of connected sections despite all the bolt holes were 'perfect-fitted' to 16 mm diameter bolts.

TEST RESULTS

Three different modes of failure were identified among the eight tests:

- BFcsw Bearing failure in section web around bolt hole, as shown in Figure 2.
- FFgp Flexural failure of hot rolled steel gusset plate.
- FFcs Flexural failure of connected cold-formed steel section, as shown in Figure 3.

Figure 4 presents the typical lateral load - lateral deflection curves of the test specimens and the typical moment - rotation curves of the test specimens are presented in Figure 5.

The maximum moment resistances of the test specimens may be evaluated at two locations, namely, at the centreline and at the failure position of the connections for different purposes. The centreline evaluation enables easy comparison of the moment capacities of the connections against applied moment obtained directly from conventional structural analysis while the failure position evaluation is required for connection design.

In general, the moment resistances are first evaluated at the centreline of the connections, and the level arm coefficients defined in Figure 6 are then applied according to the associated failure modes to give the moment resistances of the connections at the failure positions.

The results of all the eight tests are summarized in Table 1 together with the measured material properties and dimensions of the cold-formed steel sections and the gusset plates.

In order to assess the effectiveness of the bolted moment connections, a moment resistance ratio, Ψ , is established which is defined as follows:

$$\Psi = \frac{\text{Measured moment resistance of a connection}}{\text{Measured moment capacity of connected section}}$$

All the measured moment resistances are normalized with the ratio of design yield strength and design thickness to measured yield strength and measured thickness of the test specimens. For test specimens with excessive deformation under testing, the moment resistance of the connection is restricted to be the applied moment at a connection rotation of 0.05 radian.

COMPARISON AND DISCUSSION

After data analysis, the moment rotation curves of the internal and the external beam-column sub-frames are presented in Figures 7 and 8 for easy of comparison. The results of all the tests are compared among each other and the findings are presented as follows:

a) Tests S090A1, S180A1 and S240A1

In these three tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests S090A1, S180A1 and S240A1 at the centreline of the beam-column connections were 6.72 kNm, 16.74 kNm and 15.01 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. In test S090A1, significant bearing deformation was observed in the bolt holes of the beam members due to high moment acting at small lever arms. For both tests S180A1 and S240A1, gross bending deformation was apparent in the hot-rolled steel gusset plates.

The moment resistance ratios of the connections in tests S090A1, S180A1 and S240A1 were found to be 0.36, 0.69 and 0.74 respectively; the sections in test S180A1 is C15020 DS while the sections in the other two are C15016 DS.

b) Tests E180C1 and E240C1

In these two tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests E180C1 and E240C1 at the centreline of the beam-column connections were 20.12 kNm and 20.97 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. Gross bending deformation was apparent in the hot-rolled steel gusset plates of both tests.

The moment resistance ratios of the connections in tests E180C1 and E240C1 were found to be 0.84 and 0.88 respectively.

c) Tests S090A2, S180A2 and E180C2
In these three tests, large deflections and rotations of the test specimens were observed during load application. The measured moment resistances of tests S090A2, S180A2 and E180C2 at the centreline of the beam-column connections were 18.59 kNm, 23.16 kNm and 24.44 kNm respectively. There was no distinctive out-of-plane deformation of the test specimens during testing. After the tests, all the members of the sub-frames were disassembled from the connections for inspection. In test S090A2, significant deformation was observed in the bolt holes of the beam members due to high moment acting at small lever arms. For both tests S180A2 and S240A2, flexural failure in connected cold-formed steel sections was apparent.

The moment resistance ratios of the connections in tests S090A2, S180A2 and E180C2 were found to be 0.57, 0.92 and 0.97 respectively.

d) For internal beam-column sub-frames with 10 mm thick gusset plates, the increase in the bolt pitch from 90 mm to 180 mm and then to 240 mm is shown to increase the moment resistance ratio of the proposed connection configuration from 0.36, to 0.69 and then to 0.74, as shown in tests S090A1, S180A1 and S240A1. Moreover, for external beam-column sub-frames with 10 mm thick gusset plates, the increase in the bolt pitch from 180 mm to 240 mm is also shown to increase the moment resistance ratio of the proposed connection configuration from 0.84 to 0.88 as shown in tests E180C1 and E240C1.

This shows that connections with large bolt pitch will always give high moment resistances. Moreover, with an increase in the moment resistances, flexural failure in the gusset plates rather than bearing failure in the connected section web becomes critical in the connections.

e) By increasing the thickness of gusset plate from 10 mm to 16 mm, the moment resistance ratio of the proposed connection configuration is found to be increased from

- 0.36 to 0.57 as shown in tests S090A1 and S090A2,
- 0.69 to 0.92 as shown in tests S180A1 and S180A2, and
- 0.84 to 0.97 as shown in tests E180C1 and E180C2.

This shows that thicker gusset plates will always give high moment resistances. Moreover, with an increase in the moment resistances of the gusset plates, flexural failure in the connected cold-formed steel sections rather than flexural failure of the gusset plates becomes critical in the connections. The maximum moment resistance of the proposed connection configuration is found to be over 90% of the moment capacities of the sections, demonstrating that the proposed connection configuration is effective in transferring moment across the connected members.

f) As a bolt pitch of 90 mm is found to give low moment resistance with large connection rotation and member deformation, it is thus not recommended to be used in moment connections.

CONCLUSIONS

In order to enable moment framings for cold-formed steel sections in building applications, a bolted moment connection configuration for double lipped C sections back-to-back was proposed. A total of eight internal and external beam-column sub-frame tests were executed and three different modes of failure were identified in the tests. The moment resistances of the proposed connection configuration were found to range from 36% to 97% of the moment capacities of the connected sections. Consequently, it is demonstrated that bolted moment connections are structurally feasible and economical. Furthermore, structural members with double lipped C sections back-to-back are shown to be practical in building short to medium span portal frames with bolted moment connections through rational design.

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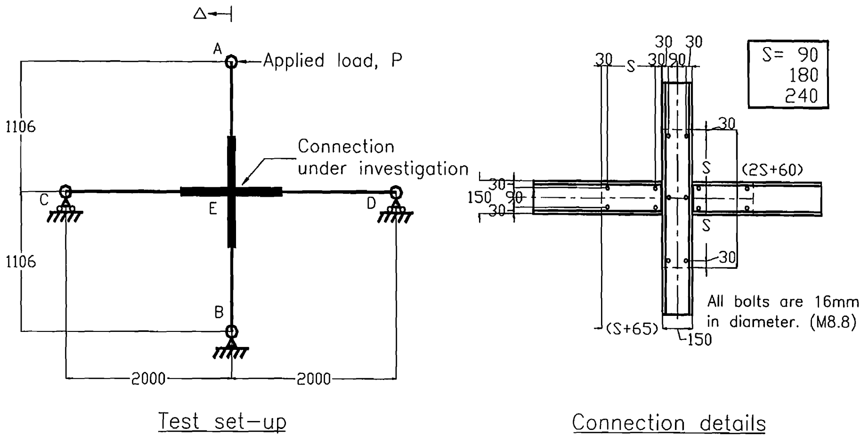
Table 1 Summary of test program and test data

Test	Section	Maximum applied force (kN)	Failure mode	Maximum moment resistance: Moment (kNm)	Maximum moment resistance: Rotation (rad.)	Moment resistance at 0.05 rad (kNm)	Member: Thickness (mm)	Member: Yield strength (N/mm^2)	Gusset plate: Thickness (mm)	Gusset plate: Yield strength (N/mm^2)	Centreline of connection: Normalised moment (kNm)	Centreline of connection: ψ	Failure position of connection: Normalised moment (kNm)	Failure position of connection: ψ (CFS)	Failure position of connection: ψ (HRS)
S090A1	C15016DS	6.08	BFcsw	6.72	0.040	-	1.62	471	9.87	388	6.18	0.36	6.18	0.36	-
S090A2	C15020DS	16.81	BFcsw	18.59	0.075	12.00	1.94	459	15.89	411	12.13	0.57	12.13	0.57	-
S180A1	C15020DS	15.14	BFcsw/FFgp	16.74	0.040	-	1.98	451	9.87	388	15.25	0.71	14.68	0.69	0.75
E180C1	C15020DS	9.10	FFgp	20.12	0.030	-	1.95	454	9.87	388	18.61	0.87	17.91	0.84	0.90
S180A2	C15020DS	20.94	FFcs (CFS)	23.16	0.031	-	1.99	454	15.89	411	23.07	1.08	19.72	0.92	-
E180C2	C15020DS	11.05	FFcs (CFS)	24.44	0.012	-	1.94	468	15.89	411	24.23	1.13	20.71	0.97	-
S240A1	C15016DS	13.57	FFgp	15.01	0.010	-	1.63	465	9.87	388	12.96	0.76	12.47	0.74	0.67
E240C1	C15020DS	9.48	FFgp	20.97	0.041	-	1.94	456	9.87	388	19.50	0.91	18.77	0.88	0.94

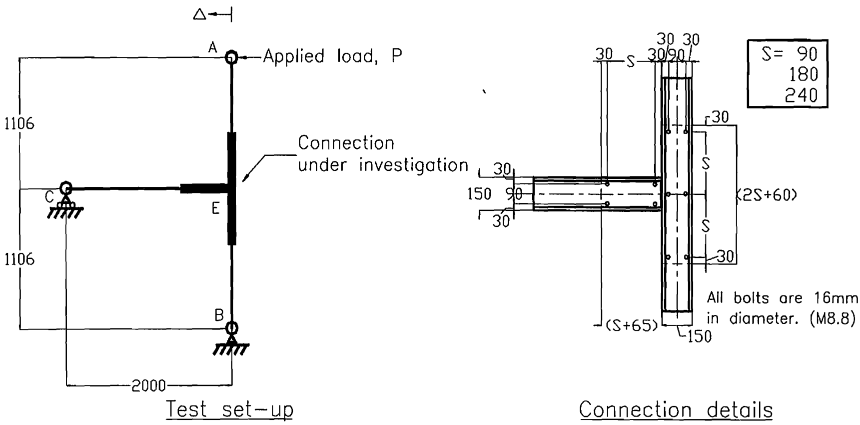
Notes:

S denotes an internal beam-column sub-frames with a 'cross' shaped gusset plate under lateral load
E denotes an external beam-column sub-frames with a 'tee' shaped gusset plate under lateral load
90 denotes a bolt pitch of 90 mm
180 denotes a bolt pitch of 180 mm
240 denotes a bolt pitch of 240 mm
A denotes 4 bolts per member
C denotes 4 bolts per member (same as A)
1 denotes the thickness of gusset plate thickness at 10 mm
2 denotes the thickness of gusset plate thickness at 16 mm

The measured moment capacities of C15016DS G450 and C15020DS G450 are 16.95 kNm and 21.36 kNm respectively.



a) Internal beam-column sub-frames.



b) External beam-column sub-frames.

Figure 1 General arrangement of beam-column sub-frames.

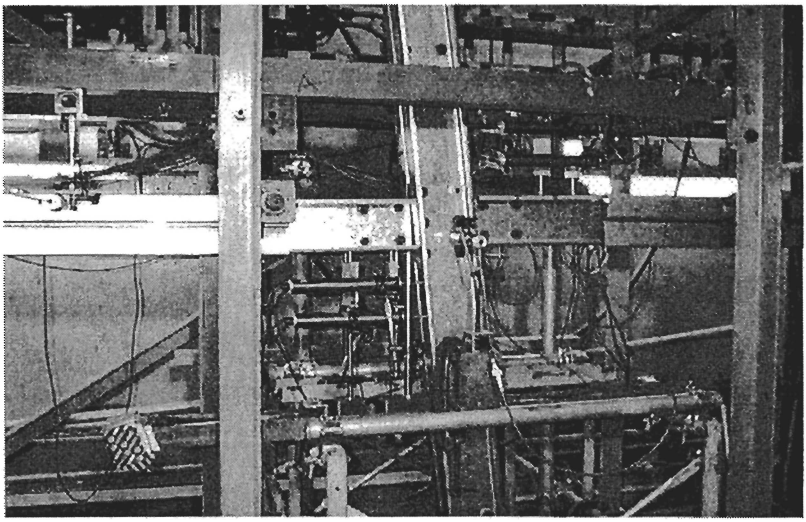


Figure 2 BFcsw Bearing failure in section web around bolt hole

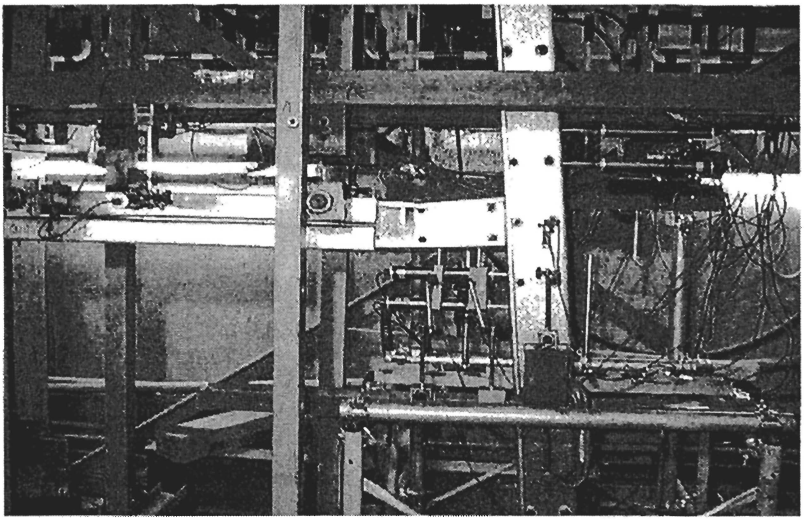


Figure 3 FFcs Flexural failure of connected cold-formed steel section

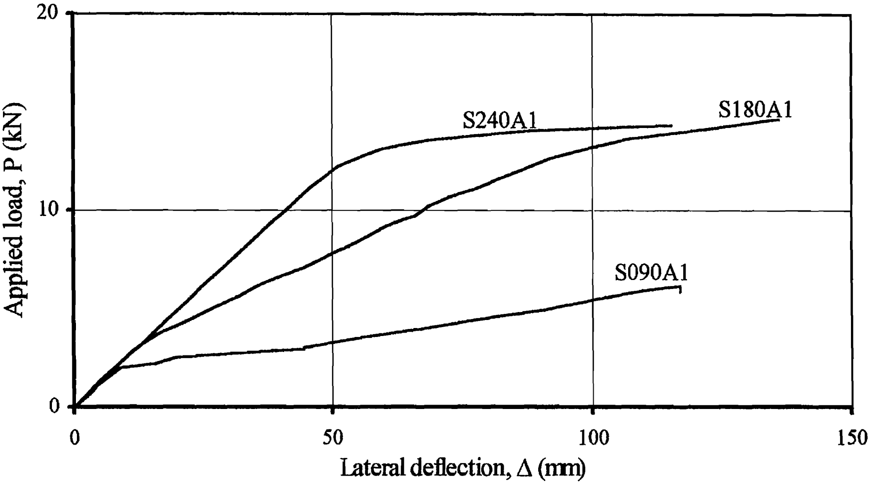


Figure 4 Load deflection curves of tests S090A1, S180A1 and S240A1.

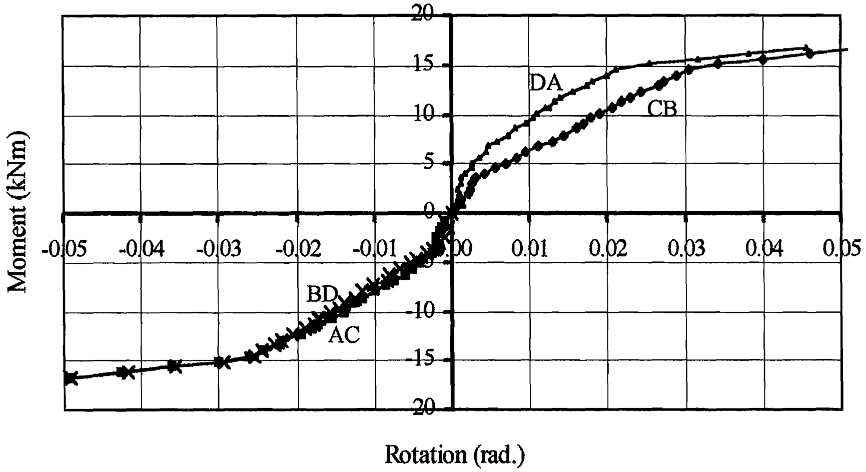
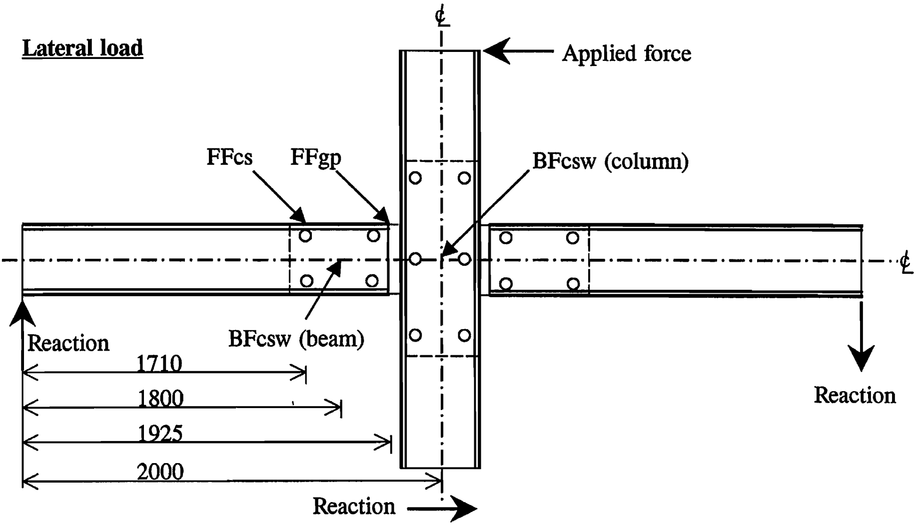


Figure 5 Moment rotation curves of test S180A1.



Mode of failure	Level arm coefficient
FFcs	1710/2000 = 0.86
FFgp	1925/2000 = 0.96
BFcsw (beam)	1800/2000 = 0.90
BFcsw (column)	2000/2000 = 1.00

Figure 6 Level arm coefficient in lateral loading tests.

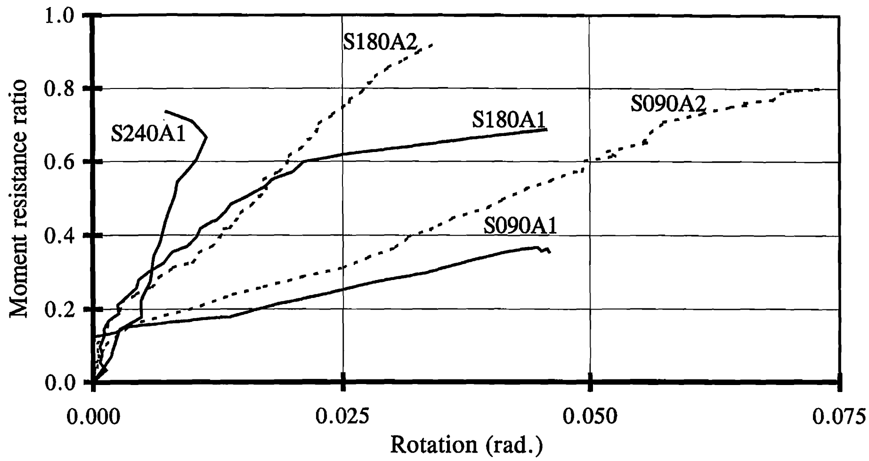


Figure 7 Moment rotation curves of internal beam-column sub-frame tests.

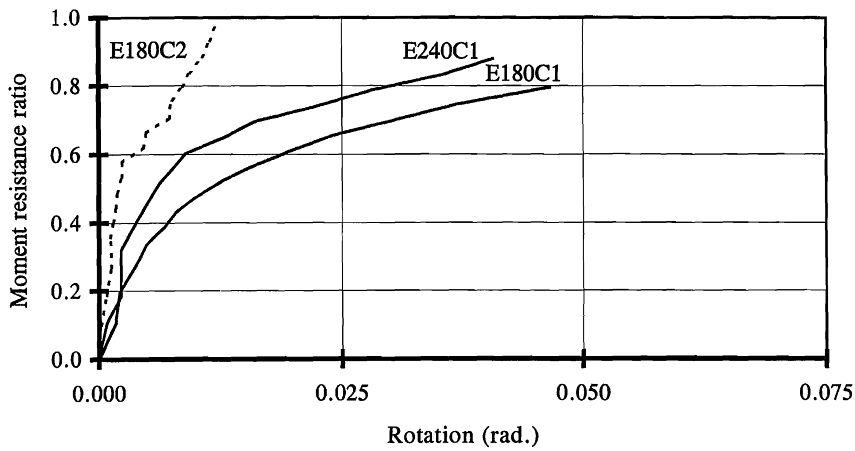


Figure 8 Moment rotation curves of external beam-column sub-frame tests.