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BEHAVIOUR OF COLD-FORMED STEEL SEMI RIGID CONNECTIONS

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Abstract

Ductility and inelastic performance are important considerations in aseismic design of buildings. The dissipation of energy due to inelastic deformation is predominantly required in the connections like beam column joints. It is necessary to design these joints as semi rigid for its economic and structural benefits. Semi-rigid connections have highly nonlinear behaviour that makes the analysis and design of frames difficult and complicated. Steel structures are highly regarded for their seismic performance. It is required to understand and study the inelastic behavior of steel connections which would help in an economical and simpler design. This paper involves the modeling of deformational behaviour of a cold formed steel connection in a finite element software simulating the real time behavior. The ultimate moment and rotation is studied for different semi rigid connections after validation of the model.

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Introduction

Aseismic design of buildings is being the most researched area since moment resisting steel frames are highly regarded for their seismic performance. This regard is based on their ductility and inelastic performance, since inelastic deformation is used to dissipate energy during major earthquakes. This dissipation of energy is predominantly required in the connections like beam column joints. The internal forces and moments produced in these connections influence the behaviour of the overall structure. It is required to understand and study these internal forces and therefore the inelastic behaviour. Such a study of the inelastic behaviour of steel connections would help in an economical and simpler design of connections in steel frames.

To achieve an economical design, usually it is important that the connections develop the full strength of the members. Usually connection failure is not as ductile as that of steel member failure. Hence it is desirable to avoid connection failure before the member failure. Therefore, design of connections is an integral and important part of design of steel structures. They are also critical components of steel structures, since they have the potential for greater variability in behavior and strength, they are more complex to design than members, and they are usually the most vulnerable components, failure of which may lead to the failure of the whole structure.

The beam – column connections are classified on the basis of load transfer mechanism as follows, Simple or flexible connection. (Transfers Shear), Moment resisting or Rigid connection. (Transfers moment) and Semi rigid connection. (Transfers Shear and Moment)

Semi-rigid connections are connections that have a dependable and known moment capacity intermediate in degree between the rigidity of rigid connections and the flexibility of simple shear connections. They possess an insufficient rigidity to hold the original angles between the connected members. Semi-rigid connections are designed to resist shear and moments whose values are intermediate between the values for simple and fully rigid connections. Semi rigid connections do not have sufficient stiffness to maintain the angle between the intersecting members. Flexible Moment Connections are also known as Semi-Rigid connections.

Literature Review

Bayan Anwer et al (2012) did the Structural Performance of Bolted Moment Connections among Single Cold-Formed Channel Sections. This paper presents an experimental investigation on bolted moment connections between single cold-formed channels connected back-to-back at the joints. A total of ten column-base connection tests and beam-column sub frame tests with different

connection configurations were conducted to investigate the performance of the connections in term of strength and stiffness.

The beam and column members are formed from single cold-formed lipped channel sections. The flange width, web and lip depth of the cold-formed section are 50,100 and 14 mm, respectively, and the thickness of each section is either 1.6 or 2 mm. The connection test specimens consisted of column-base and beam-column sub frame tests formed by single cold-formed channels C10016 or C10020 with a member thickness of 1.6 and 2 mm respectively. For all specimens, bolts grade 8.8 of 12 mm diameter were used. A total of eight column-base connections were investigated to examine the effect of bolt arrangement on structural performance of the connections for two different member thicknesses. The test specimens referred as CB02, CB03, CB04, where a column member is connected to a typical fabricated steel base plate with two, three and four bolts respectively. Another two isolated beam-column connection tests were carried out under lateral load to assess the strength and stiffness of the connections with different member thickness designated as BC-T1.6 and BC-T2. Channel sections were connected back to back at the joint. Fig 1 shows the typical setup used.



Fig.1 Experimental Test Setup

Table 1 and fig 2 gives the summary of test results.

Table 1. Summary of Test Results of beam column Test specimens.

| Beam Column Test Reference | Applied Load (kN)(kips) | Measured Moment of Resistance (kN.m)(kip.ft) | Rotational Stiffness (kN.m/rad) |
|----------------------------|-------------------------|--|---------------------------------|
| BC-T1.6 | 7.8(1.75) | 3.94(2.90) | 54 |
| BC-T2.0 | 8.7(1.95) | 4.2(3.09) | 56 |

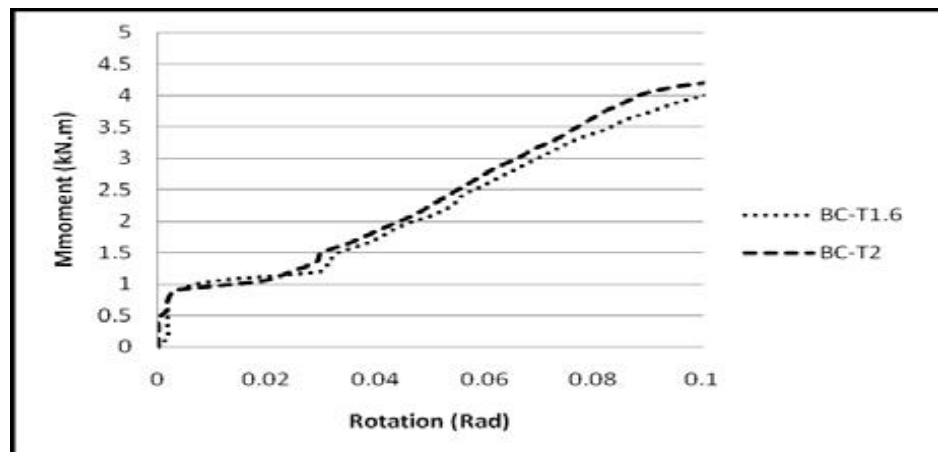


Fig.2 Moment-rotation curves of beam-column connections.

In beam-column connection test specimens, the moment capacity of joint compared to the moment capacity of beam section were 0.85 and 0.70 with rotational stiffness estimated 54 and 56 kNm/rad for 1.6 and 2 mm thick sections respectively. Consequently, simple, practical and effective joints are achieved to be used among cold-formed structural framing.

Validation of Analytic Work

The finite element method is now widely used and is a well accepted tool for accurately simulating complex structural systems, relatively few finite element analyses of storage racks have been reported in the literature. In this study, finite element models have been used to determine the stiffness and strength of bolted beam column connections under static loading. The advantage of FEM as compared with finite difference methods is that complicated geometry, general boundary conditions and variable or non-linear material properties can be handled relatively easily. Finite Element models were developed using the ABAQUS software.

Beam of Channel section of dimensions 100 mm x 50 mm x 2 mm with lip of 14 mm and Column of Channel section of dimensions 100 mm x 50 mm x 2 mm with lip of 14 mm were created in ABAQUS as shown in figure 3 and figure 4.

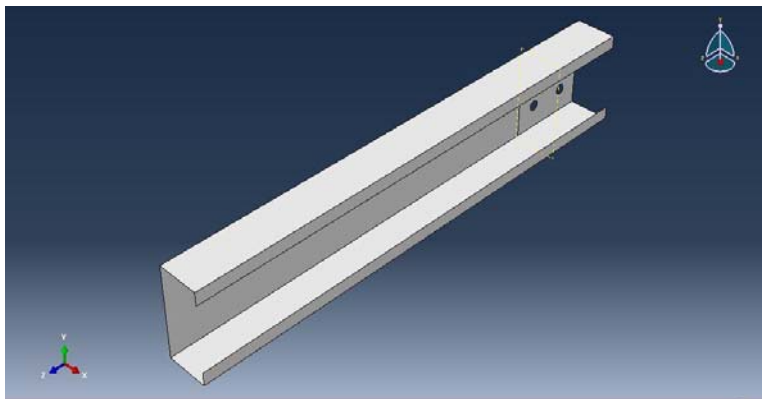


Fig. 3 Creation of Beam

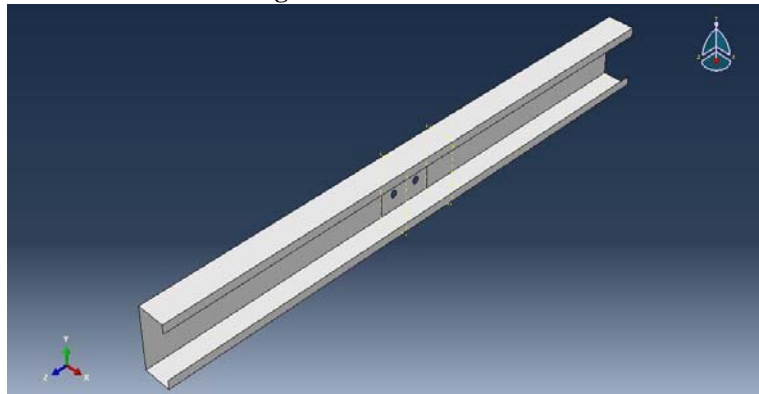
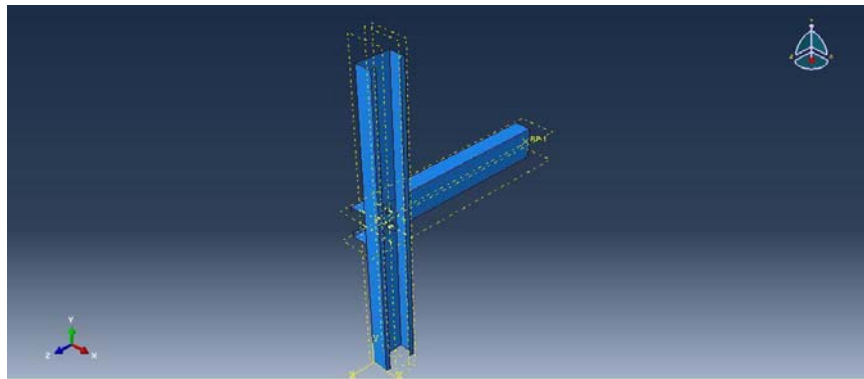


Fig. 4 Creation of Column

For the beam, column section, density and Youngs modulus were 7850 kg/m^3 and $2.1 \times 10^5 \text{ N/mm}^2$ used with a yield stress of 250 N/mm^2 . Poisson ratio for the assembly was assigned as 0.3. After Assigning the material property for the model, the color of the model changes. The parts are then assembled as shown in fig 5.

**Fig. 5 Assembly of model**

General static loading was given at the CG of the beam channel section at the end. For applying load at the end, Reference point was created at the CG point. The following figure6 shows the load applied at the end of the beam.

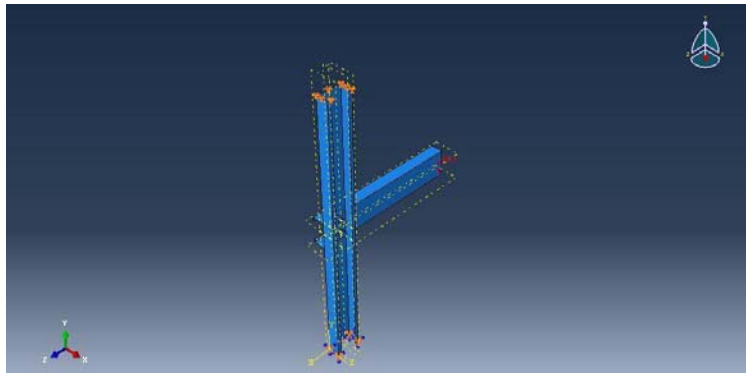
**Fig. 6 Applying Static load**

Table 2 shows the validation results

Table 2. Comparison of Experimental and Analytical Results.

| Parameters | Experimental work in literature | Analytical work | % Variation |
|----------------------|---------------------------------|---------------------|-------------|
| Moment of Resistance | 4.2 kNm(3.09 kip.ft) | 4.5kNm(3.31 kip.ft) | 7.1 |
| Deflection | 40 mm(1.57 in) | 36 mm(1.41 in) | 10 |

Moment Curvature Study

After the validation, the study has been extended to other semi rigid configurations.Length of the beam is chosen as 1000 mm and the length of the columns is 3000 mm. The beam & column are modelled three dimensional deformable types and base feature is shell. The beam and column size is 100 mm x 50 mm x 5 mm. The size of angle section used to connect the beam and column is 40 mm x 40 mm x 3.15 mm.For the beam, column and angle section, density and Youngs modulus were 7850 kg/m³ and 2.1 x 10⁵ N/mm² used with a yield stress of 250 N/mm². Poisson ratio for the assembly was assigned as 0.3.

The assembly of model, the individual parts such as beam, column and angle are assembled with help of module in the assembly section. The finite element model consists of a column fixed in the base and pinned in the top.In the assembly section independent instance type is chosen and this is mesh on instance another type is mesh on part. The column is fixed at base and pinned at top. The beam is connected with the column at one end and the other end is free. At the free end, the load is to be applied.

JOINT CONFIGURATIONS

The different joint configurations for analytical study are assembled as shown in the Table 3 shows the details of the joint configurations.

Table 3 Beam column Joint configurations.

| S.No. | Joint Section | Description | No.of bolts Used | No.of Angles |
|-------|---------------|--|------------------|--------------|
| 1 | J 1 | Bolt only connection | 4 | 0 |
| 2 | J 2 | Web angle connection | 8 | 1 |
| 3 | J 3 | Top And Seat angle Connection | 12 | 2 |
| 4 | J 4 | Web Angle Plus top and seat angle connection | 16 | 3 |

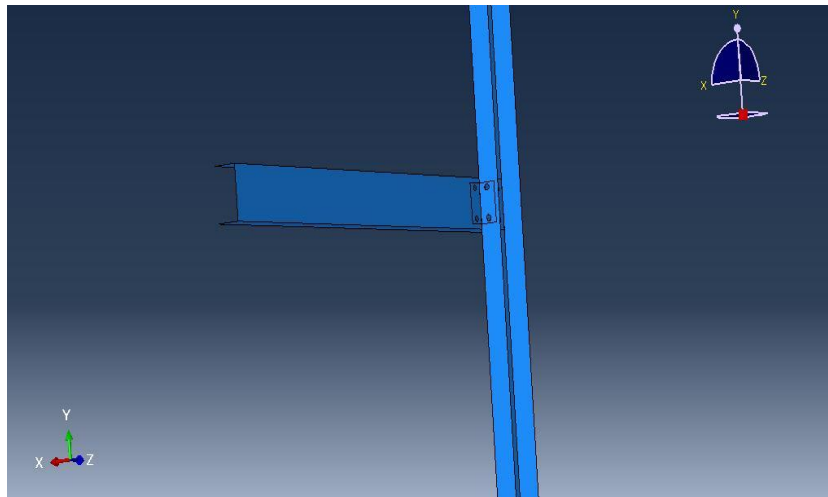


Fig.7 Joint 1 (Bolt only Connection)

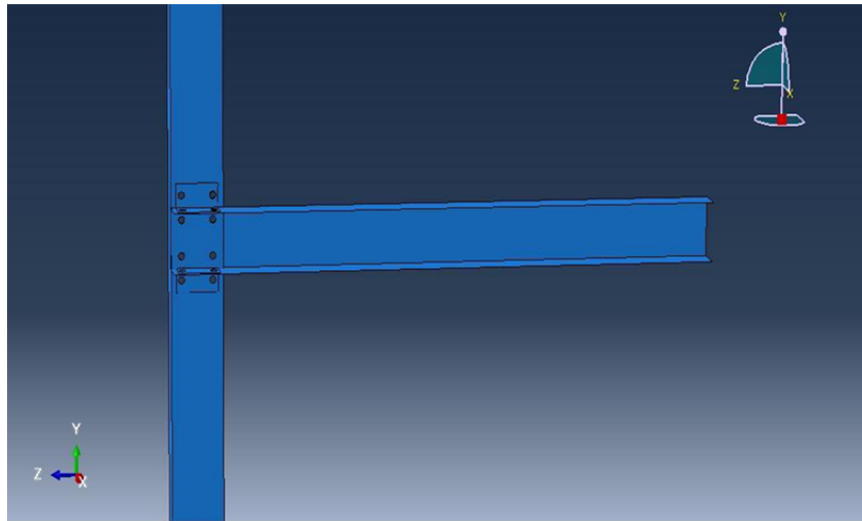


Fig.8 Joint 2 (Web Angle Connection)

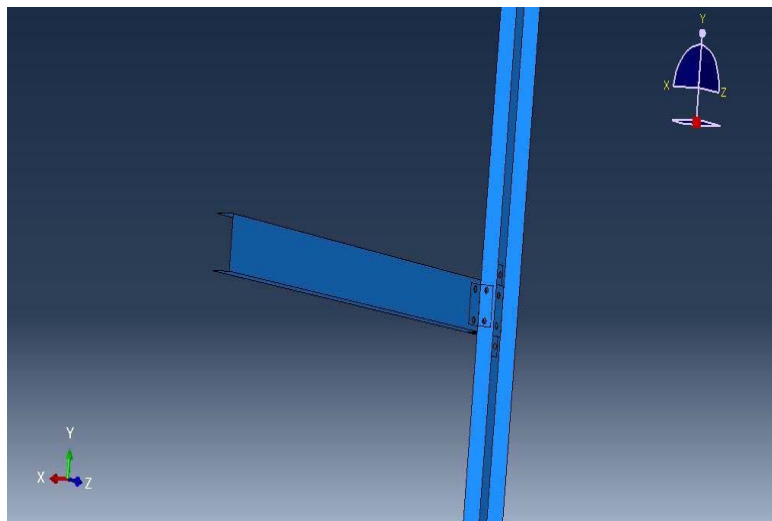


Fig.9 Joint 3 (Top and Seat Angle Connection)

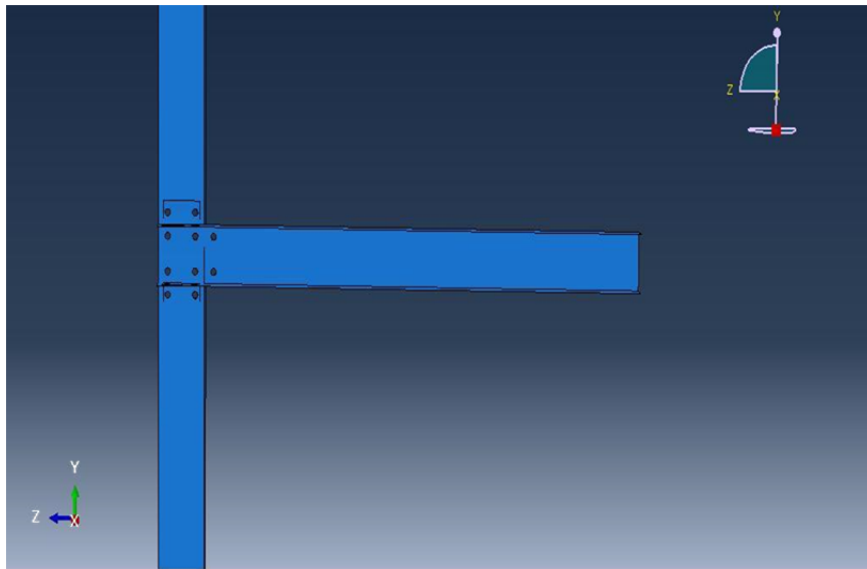


Fig.10 Joint 4 (Web Angle Plus Top and Seat Angle Connection)

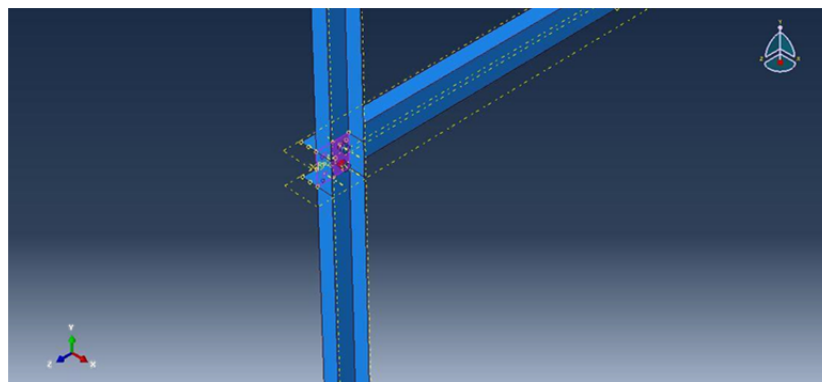
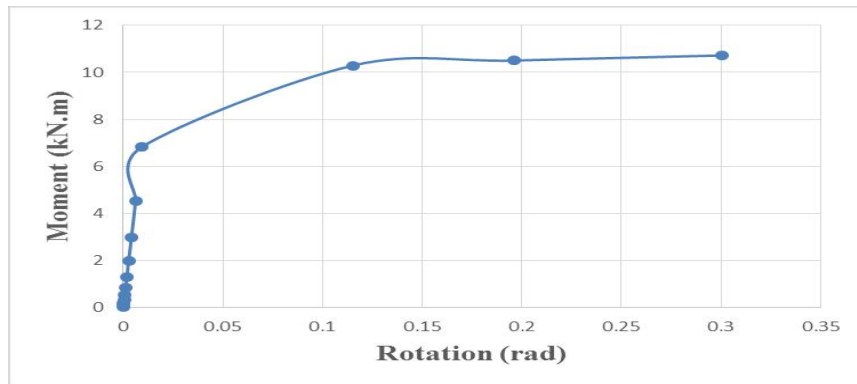


Fig.11 Joint 4 (Web Angle Plus Top and Seat Angle Connection Frontview)

**Table 4 Results of Beam column Joint (J1).
(BOLT ONLY CONNECTION)**

| Load (kN)(kips) | Deflection (mm)(in) | Moment (kN.m)(kip.ft) | Rotation(rad) |
|-----------------|---------------------|-----------------------|---------------|
| 0 (0) | 0 (0) | 0 (0) | 0 |
| 0.1 (0.022) | 1.41E-01 | 0.09 (0.066) | 1.24E-04 |
| 0.2 (0.044) | 2.81E-01 | 0.18 (0.132) | 2.48E-04 |
| 0.35 (0.078) | 0.492471 (0.019) | 0.315 (0.232) | 0.000433223 |
| 0.575 (0.129) | 0.80906 (0.031) | 0.5175 (0.381) | 0.000711724 |
| 0.9125 (0.205) | 1.28394 (0.05) | 0.82125 (0.605) | 0.00112948 |
| 1.41875 (0.318) | 1.99627 (0.078) | 1.276875 (0.941) | 0.0017561 |
| 2.17812 (0.489) | 3.06475 (0.12) | 1.960308 (1.445) | 0.00269604 |
| 3.31719 (0.745) | 4.66748 (0.183) | 2.985471 (2.201) | 0.00410595 |
| 5.02578 (1.129) | 7.07158 (0.278) | 4.523202 (3.336) | 0.00622082 |
| 7.58867 (1.706) | 10.7257 (0.422) | 6.829803 (5.037) | 0.00944662 |
| 11.433 (2.57) | 30.271 (1.191) | 10.2897 (7.589) | 0.115084 |



Load Vs Deflection

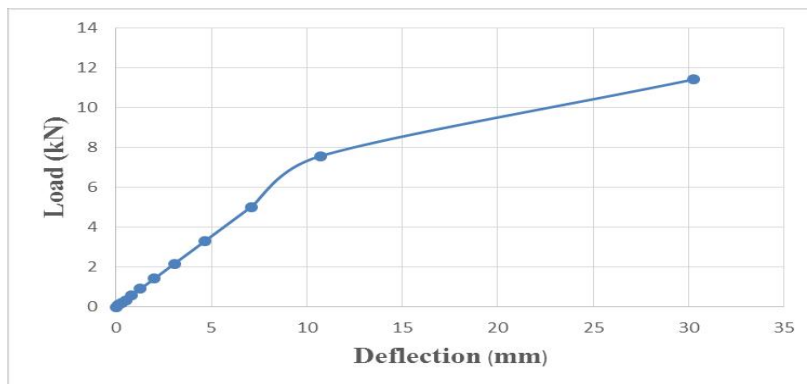


Fig.14 Load Vs Deflection Plot**Conclusion**

Joint J1 attained its maximum moment resistance of 10 kNm at a rotation of 0.10 radian and it attains 30.27 mm deflection at the 11.4 kN load.

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