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# Cyclic behaviour of bolted cold-formed steel moment connections: FE modelling including slip

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ABSTRACT

In this paper a Finite Element (FE) procedure is described for simulating hysteretic moment–rotation behaviour and failure deformations of bolted cold-formed steel (CFS) moment connections. One of the main challenges in modelling the response of bolted connections is the presence of bolt slip. A series of six beam–column assemblies comprising CFS curved flange beams, a support column and a through plate were tested under cyclic loading. The moment–rotation behaviour of the connections was dominated either by flexure in the beams or by bolt slip in the connections. FE models presented in this paper incorporate geometrical imperfections of the beams, material properties obtained from tensile coupon tests and bolt slip to address these two types of behaviour. The updated FE models result in an accurate prediction of the hysteretic moment–rotation behaviour of the connections dominated by a flexural behaviour in the beams. A simplified cyclic slip model which allows slip at a specified reduced slip resistance load simulates reasonably well the hysteretic behaviour of connections dominated by bolt slip. Using the updated FE model, the failure modes of the connections predicted by the simulations also agree well with those observed in the experiments.



## 1. Introduction

Analytical investigations of cyclic behaviour of bolted moment resisting connections with slip-bearing action are relatively limited [1,2]. After initiation of connection slip, moment–rotation behaviour becomes highly nonlinear due to the rather unpredictable slip–grip action of the bolts on the surfaces of connected plates, which eventually leads to contact and elongation of bolt holes [3,4]. Activation of connection slip can be highly beneficial in dissipating seismic energy, by achieving stable hysteretic cycles and postponing plastic deformations and failures in the connected members [3,4]. This is particularly important in cold-formed steel (CFS) structures comprising thin-walled elements which are vulnerable to premature local buckling [4]. Extensive FE analyses and experiments were conducted by the authors [4–8] to develop a ductile CFS moment resistant beam-to-column connection as schematically shown in Fig. 1. The beam section is formed with curved flanges which increase local buckling resistance and avoid the large flat flange width/thickness ratios found in conventional CFS beam sections such as back-to-back channel, and lipped or unlipped sections. As previously presented by the authors [6,7], the curved flange section has been arrived at through a step-by-step process of increasing the number of flange bends in the section, shown in Fig. 2(a). This process leads to improvement in the moment–rotation characteristics of the CFS beams, as seen in Fig. 2(b); $M_p$ is the nominal plastic moment of the beam F0, for nominal yield strength of 275 MPa. Eventually, the predominant bucking mode is shifted from the flanges in the flat flange sections to the webs in the curved flange sections as seen in Fig. 2(c).

The through plates are the main components of the connection, transferring the forces to both near and far sides of the column by in-plane action. It was shown by both FE analyses and experiments that sets of out-of-plane stiffeners are needed (shown in Fig. 1) to further improve the ductility and energy dissipation capacity of the beam–column assembly [4,6–8].

Two types of moment–rotation behaviour were identified in the experiments of the new CFS moment connections [4]. In the first type flexural deformation and local buckling in the beams dominated the rotational behaviour, whilst in the second type the overall behaviour was dominated by a connection slip-bearing action. The FE models of the beam–column assembly were updated to address these two types of behaviour. Monotonic FE analyses presented elsewhere by the authors [7] showed good agreement with the envelope of cyclic moment–rotation curves of the first type of connections. The accuracy of the FE results was enhanced by including geometric imperfections of the beams, material properties obtained from tensile coupon tests and connection slip. The local slip occurring in the bolts of the connections was ignored in order to achieve a simple and practical FE model for cyclic loading.

Two common methods can be used to simulate bolts in FE models. The more complex approach is to model the body of the bolts and simulates the slip-bearing action by applying clamping forces to mobilise frictional

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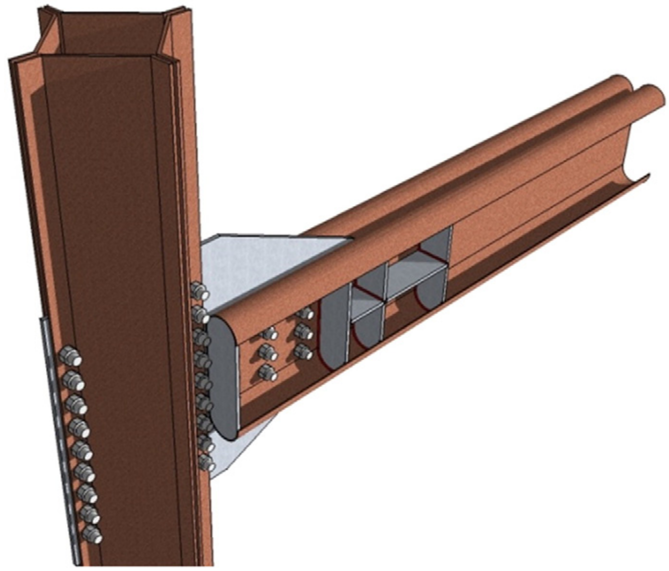

**Fig 1.** Schematic drawing of the new CFS moment resistant beam–column connections with curved flange beam, diamond column, through plate and out-of-plane stiffeners [4–8].

resistance and contact interactions between the connected surfaces [9,10]. This method is mainly used for monotonic loading since it is computationally too time consuming for cyclic loading, especially for large models. A simpler approach is to use connector elements between the nodes of the connected plates in place of the bolts [11,12]. Appropriate elasto-plastic behaviour needs to be defined for the connector elements to represent the actual slip-bearing action. Since the simplified method normally causes less convergence problems than the complex method it is adopted in this paper as computationally appropriate for cyclic loading. This can be further justified by the fact that inherent uncertainties in the slip-bearing action and highly nonlinear behaviour of bolted connections during cyclic loading limit the accuracy of the complex method.

## 2. Experimental set-up and specimen details

The experimental investigation was carried out by cyclic testing of six beam-to-column assemblies with nominal dimensions shown in Fig. 3, with (A) 3 mm and (B) 4 mm thickness beams, each with three configurations of out-of-plane stiffeners (A1–3 and B1–3) [4]. Specimens A1 and B1 had no out-of-plane stiffeners. Specimens A2 and B2 were provided with a "minimum" configuration [7,8] of vertical stiffeners at both beam-end and through-plate-end. Specimens A3 and B3 had a full set of stiffeners (Fig. 3), in an "optimum configuration" (studied previously by the authors [6–8]). A stub column was fixed to the reaction frame (Fig. 4) and cyclic loading was applied through a pinned connection at the beam-end (Figs. 3 and 4). The loading protocol was in accordance with the AISC Seismic Provisions [13] for qualifying beam–column moment connections. As specified in the Provisions [13], lateral restraints were used at the expected plastic hinge region and at the concentrated loading region.

For all six specimens, the requirements for slip-critical joints specified in the AISC Specification for Structural Joints [14] were used by assuming uniform rotation in the beam-to-through plate and the through plate-to-column connections. The ratio of the required resistance of the farthest bolts over the design resistance for specimen type A was $R_{req}/R_n = 0.83$, but this ratio was $R_{req}/R_n = 1.11$ for specimen type B. Therefore, if the beam plastic moment capacity was mobilised in the tests of specimen type B, slip in the connections could be triggered.

The stress–strain characteristics obtained from tensile coupon tests on steel plates (conforming to ASTM A370-02 [15]) were modelled by using trilinear elastic–plastic curves with strain hardening and used for the FE models of the beams of all six specimens (Fig. 5). True stresses and strains ($\sigma_{true}$, $\varepsilon_{true}$) were derived from the engineering stresses ($\sigma$) and strains ($\varepsilon$) to account for the reduced area due to necking in the tensile coupon tests and hence, $\varepsilon_{plastic}$ is the true plastic strain. The basic material properties of the through plates are given separately in Table 1.

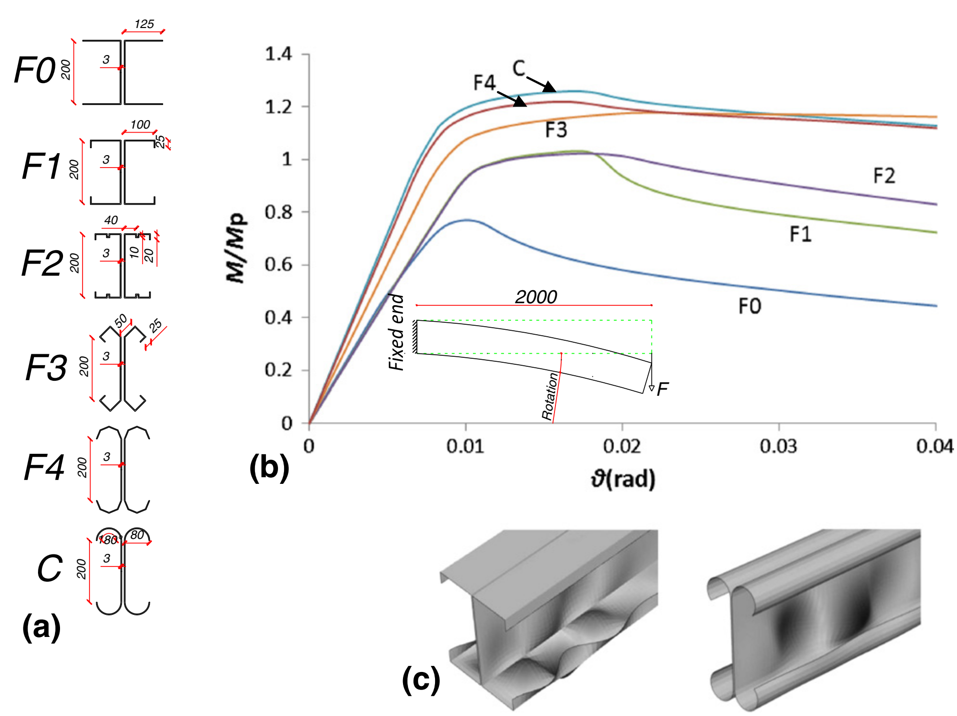


**Fig. 2.** Comparison of beams with different cross sections: (a) section shapes, (b) moment–rotation diagrams for fixed end beams with cross sections F0 to F4 and C, and (c) predominant buckling modes of sections F1 and C [6,7].