Cold-formed Steel Pitched-roof Portal Frames of Back-to-back Plain Channel Sections and Bolted Joints

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Abstract

The paper summarises the results of an experimental program carried out in order to evaluate performance of pitched roof cold-formed steel portal frames of back-to-back plain channel sections and bolted joints. Three different configurations of apex and knee joints were tested. The behaviour and failure mechanisms of joints were observed in order to evaluate their stiffness, strength and ductility. Distribution of bolt forces showed to differ significantly from the classical assumption of forces proportional to the bolt centre of gravity. Joints between cold-formed members with bolts in the web only result in reduction of joint moment capacity and premature web buckling. The component method was applied in order to characterise joint stiffness and moment capacity on the purpose of frame analysis and design. To check the design procedure, full-scale tests on frames were performed. Fair agreement between analytical and experimental results was obtained.

Introduction

Previous studies by Lim and Nethercot, 2004 and Chung and Lau, 1999 showed that bolted joints in cold formed steel portal frames have a semi-rigid behaviour. Also, these types of joints are partially resistant (Lim and Nethercot 2003, Wong and Chung 2002). An important contribution to the global flexibility of the joints, besides the bearing effect (bolt hole elongation), is due to the deformation induced by the local buckling or distortion of the thin walled profiles. In an

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unwisely configured joint premature local buckling can cause the failure of the joint itself well below the expected load bearing capacity. In case of back-to-back bolted connections, when bolts are installed only on the web of cold-formed section, the local buckling is made more critical by stress concentrations, shear lag and bearing deformations around bolt holes (Dundu and Kemp 2006).

However, in case of usual cold-formed steel sections, both tests and numerical simulations show the bearing work of bolts associated wit elastic-plastic elongation of bolt-holes is by far the most important component controlling the stiffness and capacity of such type of connections (Lim and Nethercot 2004, Yu et al. 2005, Ho and Chung 2006). The contribution of other components, such as flanges in tension and compression due to bending action, and the web in shear due to transverse action is significantly lower.

Based on tests on apex and eaves bolted joints of built-up back-to-back plain channel sections (Dubina et al. 2004), which are summarised in the present paper, the component method is used to characterise their stiffness and strength (Nagy et al., 2006).

The global behaviour of cold-formed steel portal frames of bolted joints were studied experimentally by Lim (2001), Dundu and Kemp (2006), and Kwon et al. (2006). All these studies provided evidence of the crucial importance of joint performance on the global response of frames. In present paper, the stiffness and moment capacity obtained by the component method (EN1993-1-8, 2003) are used to model the global structural response. The calculation procedure is validated using full-scale tests on pitched-roof portal frames.

Summary of testing program on joint specimens

In order to be able to define realistic specimen configurations a simple pitched roof portal frame was first designed with the following configuration: span 12 m; bay 5 m; eaves height 4 m and roof angle 10°. This frame was subjected to loads common in the Romanian design practice, totalling approximately 10 kN/m uniformly distributed load on the frame. The frame was analysed and designed according to EN 1993-1-3 (2001) rules.

Elements of the portal frame resulted back-to-back built up sections made of Lindab C350/3.0 profiles (yield strength f_y=350 N/mm^2). Using these cross section dimensions, three alternative joint configurations were designed (see Figure 1 and Figure 2), using welded bracket elements (S235: f_y=235 N/mm^2).

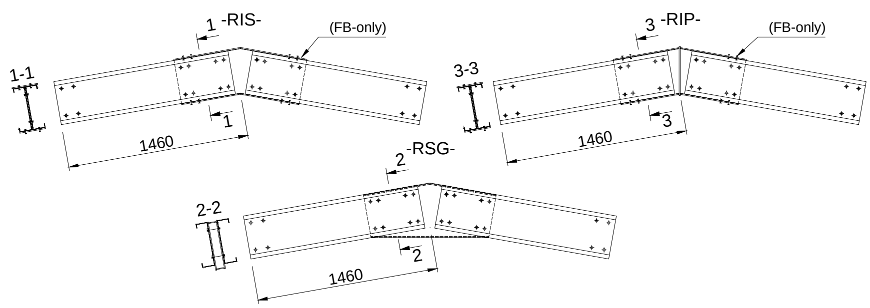


Figure 1. Configurations of ridge joints.

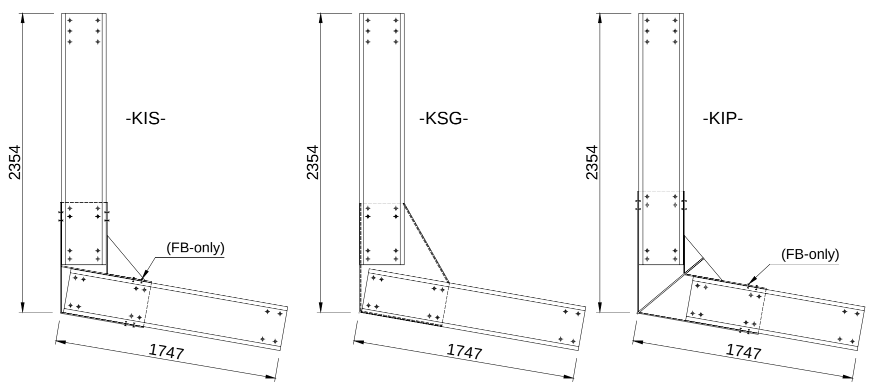


Figure 2. Configurations of knee joints.

One group of specimens (KSG and RSG) used spaced built-up gussets. In this case, bolts were provided only on the web of the C350 profile. In the other cases, where two different details were used for the connecting bracket – i.e. welded I sections only (KIS and RIS), and welded I section with plate bisector (KIP and RIP), respectively - bolts were provided on the web only, or both on the web and the flanges. Joints where bolts were provided on the web and on flanges were denoted by FB letters.

Monotonic and cyclic experiments were performed for each specimen typology, all specimens being tested statically. For monotonically loaded specimens the "yield" displacement (v_y) was determined according to the ECCS (1985) procedure. For the cyclic tests several alternative loading procedures were used: (1) the standard ECCS (1985) cyclic procedure, (2) a modified cyclic procedure, suggested by the authors, which is based on the ECCS proposal and (3) a cyclic procedure for low cycle fatigue.

Since detailed results about these tests were already reported by the authors (Dubina et al., 2004), present paper reviews only the main results.

The monotonic tests identified failure modes of the different joint typologies. All specimens had a failure due to local buckling of the cold formed profiles; however two distinctive modes were identified for specimens with flange bolts and those without. If no bolts are provided on the flange of profiles, initially minor bearing elongation of the bolt holes were observed, the failure being due to stress concentration in the vicinity of outer bolt row. The resulting concentration of compressive stress in the web of the C profile causes in the ultimate stage local buckling followed suddenly by web-induced flange buckling. This phenomenon occurred in a similar way in the case of RSG and KSG specimens. No important differences were observed between specimens where no bolts were provided on the flanges. In the case of the specimens with flange bolts, the stresses concentrated in the vicinity of the outer bolt row on the flange. In this case no initial elongation of the bolt holes were observed; the buckling was firstly initiated in the flange, and only later was extended into the web.

(a) (b)

Figure 3. Failure of knee specimens KIS-M (a) and KIS-FB-M (b).

To account for the flexibility of the bolted connection in structural analysis, two models are possible: one which considers both connections independently (see Figure 4a), and a simplified one, which considers the characteristics of the connection concentrated in one joint only (see Figure 4b). The former is believed to represent more exactly the real behaviour of the assembly, while the latter has the advantage of simplicity. Similar models can be used for knee joint configurations. Moment-rotation relationships characterising connection response were derived for both the left and right ridge connections (beam and column connection in the case of knee joins). Moments were computed at the end of the bracket. The corresponding relative rotation between the bracket and the connected element θ_C* was determined from acquired data, so as to represent both flexibility of the connection (due to bolt bearing) and post-buckling deformations in the element (Dubina et al. 2004).

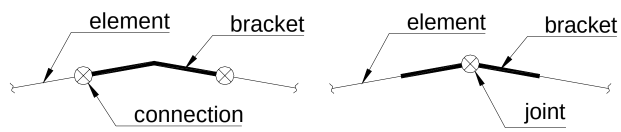


Figure 4. Two possible models for ridge joints: detailed (a) and simplified (b).

In case of the cyclic loading, the degradation of the specimens initiated with elongations in the bolt holes caused by bearing. Compared to monotonic loading, in this case the phenomenon was amplified due to the repeated and reverse loading. However, the failure occurred also by local buckling, as in case of monotonic tests, but at the repeated reversals, the buckling occurred alternately on opposite sides of the profile. This repeated loading caused the initiation of a crack at the corner of the C profile, in 2-3 cycles following the buckling, closed to the point where the first buckling wave was observed in the flange.

The component method

The component method is a general procedure for design of strength and stiffness of joints in building frames, and is implemented in EN1993-1-8, 2003. The procedure is primarily intended for heavy-gauged construction. Its application to joints connecting light-gauge members was investigated and found appropriate with minimum set of adjustments (Nagy et al, 2006). Application of the component method requires the following steps (Jaspart et al. 1999):

- identification of the active components within the joint
- evaluation of the stiffness and strength of individual components
- assembly of the components in order to evaluate stiffness and strength of the whole joint

Based on the conclusions of experimental programme, only joints with both web and flange bolts (RIS-FB-M, KIS-FB-M, and KIP-FB-M) were investigated.

Qualitative FEM simulation showed that in the case of specimens with bolts on the web only there is a stress concentration in the web, which causes premature local buckling failure. The FEM simulation also demonstrated that load distribution in the bolts is not linear. In fact, due to member flexibility and local buckling, the connected members do not behave as rigid bodies, and the centre of rotation of web bolts does not coincide with the centroid of web bolts. The centre of rotation of the connection is shifted towards the outer bolt rows (see Figure 5), whose corresponding force is an order of magnitude higher than the force in the inner bolts. Considering this observation, only the outer bolt group was considered for determination of connection characteristics using the component method.

Centre of compression of the connection was considered at the exterior flange of the cold-formed member (see Figure 5). There are a total of four bolt rows, of which three bolt rows are in the "tension" zone. The following components were identified and used to model the connection stiffness and strength:

- Cold-formed member flange and web in compression. Only strength of this component was considered, while stiffness was considered infinite
- Bolts in shear
- Bolts in bearing on the cold-formed member
- Bolts in bearing on the bracket

Stiffness and strength of all these components are readily available in EN1993-1-8 (2003), only minor adjustments being required for the case of the particular case considered here. In order to facilitate comparison with the experimental results, measured geometrical characteristics and strength (a yield strength f_y=452 N/mm^2, and a tensile strength f_u=520 N/mm^2) were considered in the case of the cold-formed member. Nominal characteristics were used for the bracket and bolt characteristics, as experimental data was not available. Partial safety factors equal to unity were considered in all cases.

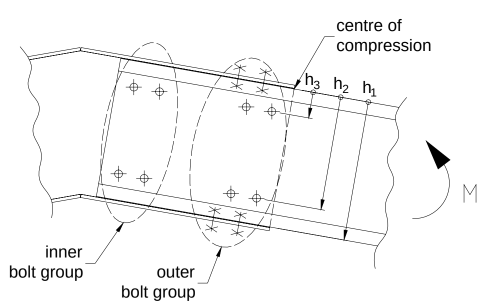


Figure 5. Bolt groups considered in analysis.

The configuration of the outer group of bolts being the same in the case of all three specimens with web and flange bolts (RIS-FB-M, KIS-FB-M, KIP-FB-M), a single set of analytical connection properties were determined. A comparison of experimental vs. analytical characteristics of connections (stiffness and moment resistance) is presented in Table 1 and Figure 6. Generally a fair agreement between experimental and analytical stiffness of the connection can be observed. Larger experimental values of stiffness can be explained by the fact that the contribution of the inner bolt group was ignored in the analytical model. Stiffness of the connection is considerably lower than the ENV1993-1-8 limits for classification of joints as rigid ($25EI_b/L_b$), which amounts to 25256 kN/m (considering the beam span L_b equal to frame span and using gross moment of inertia I_b). Therefore, these types of connections are semirigid, and their characteristics need to be taken into account in the global design of frame.

Table 1. Experimental vs. analytical connection characteristics.

Specimen	Initial stiffness, K_{iniC} [kNm/rad]		Moment resistance, M_C [kNm]	
	experimental	analytical	experimental	analytical
RIS-FB-M	6011	5224	108.0	117.8
KIS-FB-M	6432	5224	102.9	117.8
KIP-FB-M	6957	5224	116.7	117.8

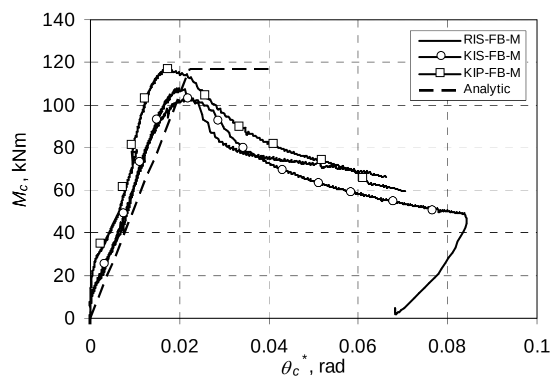


Figure 6. Experimental vs. analytical moment-rotation curves.

Moment resistance of the bolted connection $M^b_{C,Rd}$ determined by the component method amounted to 193.9 kNm, which was larger than the moment resistance of the cold-formed member $M_{beam,Rd}$, amounting 117.8 kNm. Therefore, this type of connection is a full-strength one. This was demonstrated also by the experimental results, failure mode being local buckling of the cold-formed member.

Full-scale tests of pitched-roof portal frames

Test setup

Following experimental tests on cold-formed joints, two full-scale tests on frames were performed. Frames dimensions were chosen identical to the ones in the initial design used to establish the dimensions of tested joints. Considering the poor performance of joints with web bolts only, RIS-FB and KIS-FB configurations (with both web and flange bolts) were used for frame construction. Pinned supports were used at the column bases. Objective of the full-scale tests were to assess performance of pitched-roof cold-formed portal frames with moment-resisting joints under lateral loading, with particular emphasis on earthquake loading.

The test setup consisted of two frames in upward position, located 1.5 m apart. Tie bracing was provided between the two frames in order to provide out-of plane stability. Purlins were installed on the girders, but no side rails were provided on the columns. The schematic representation of test setup is shown in Figure 7. A reaction frame was used in order to apply lateral load.

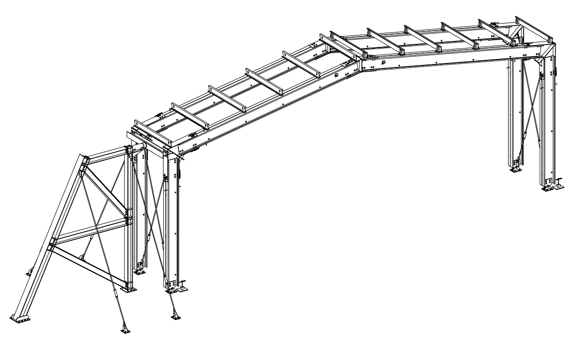


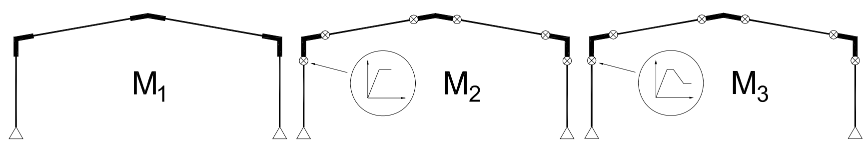
Figure 7. Experimental test setup for full-scale tests.

In the case of the first test (C1) only lateral loading was applied. For the second test (C2), gravity loading corresponding to seismic design situation (permanent and a 0.3 fraction of the snow load) was applied, followed by increasing lateral load up to failure. Total gravity loading amounted to 31.2 kN per frame, and was applied using 30 corrugated steel sheets laid on the purlins. A load cell was used in order to measure lateral load applied through a hydraulic jack. Frames were instrumented with displacement transducer to measure lateral in-plane and out-of plane displacements at the eaves, deflections at the ridge, as well as connection rotations.

Test results and comparison to numerical model

Experimental tests on ridge and eaves joints showed that bolted connections of back-to-back plain channel cold-formed members are semi-rigid, even when bolts are provided not only on the web, but also on the flanges of the channel section. Therefore, deformations can be underestimated if connections are assumed rigid for global frame analysis. In order to assess the influence of connection stiffness and post-buckling resistance, three frame models were analysed (see Figure 8). A nonlinear static analysis under increasing lateral load was applied to the models, and the results were compared to experimental ones.

The first model was a conventional model, where connections were considered rigid. Nominal geometrical characteristics were used to model members. Finite dimensions of brackets were taken into account. Local buckling of members was modelled by rigid-plastic hinges located at the extremities of cold-formed members. Analytically determined moment capacity (M_c=117.8 kNm) was considered.



(a) (b) (c)

Figure 8. Considered structural models: rigid connections - M1 (a), elastic-perfectly plastic connections - M2 (b), and degrading connections – M3 (c).

The second model (M2, see Figure 8b) was obtained from model M1 by adopting an elastic perfectly-plastic model of the connection moment-rotation response. Initial stiffness (K_{iniC}=5224 kNm/rad) and moment capacity (M_c=117.8 kNm) were the ones obtained using the analytical procedure described above (see Figure 6).

In the case of the third model (M3, see Figure 8c), post-buckling response of the connections was modelled in addition to the initial stiffness and moment capacity. Plastic rotation (plateau) was determined assuming a ductility equal to 1.5. The softening branch was determined by considering a drop of moment capacity to 50% from the maximum one, at a rotation equal to 2.5 the yield rotation (see Figure 9). The same moment-rotation characteristics were used for all connections (for both beams and columns). Influence of axial force on the stiffness and moment resistance of the connection were ignored.

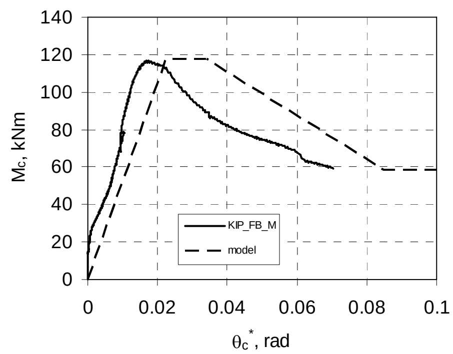


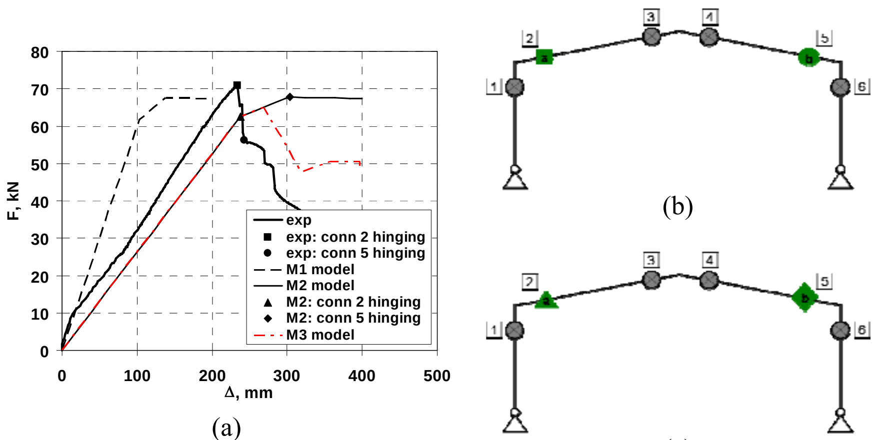
Figure 9. Experimental and analytical M3 model of connection moment-rotation relationship.

Figure 10a shows a global view of the C1 frame (tested under horizontal loading only) after the testing. Frame response during the test was characterised by a almost linear response up to the first local buckling of the beam at the connection 2 (see Figure 10b, Figure 11a, and Figure 11b), followed by a rapid loss of global frame resistance. Final collapse mechanism consisted in hinging of beam at connections 2 and 5 (see Figure 11b) near the eaves.

A comparison of the experimental and numerical lateral force – deformation curves for the C1 frame is shown in Figure 11. The force corresponds to one of the two frames from the experimental setup, assuming the force equally distributed between the two frames. It can be observed that the rigid model (M1) provides a good approximation of the initial response of the frame up to lateral forces of about 10 kN. At larger forces, models M2 and M3, with semirigid connections, provide a better approximation of the experimental response. The same pattern of member hinging as in the one observed in the experiment is obtained for the numerical model (see Figure 11c for the case of the M2 model). The M3 model captures well the post-buckling response. Both M2 and M3 models slightly underestimate global frame resistance, while overestimating lateral deformations.

(a) (b)

Figure 10. C1 frame: global view (a) and local buckling of the left beam connection (b).



(a) (b) (c)

Figure 11. Frame C1: experimental vs. numerical lateral force - deformation curves (a); position of local buckling observed experimentally (b) and in the numerical model (c).

In the case of the C2 frame, gravity loading corresponding to seismic design situation was first applied, followed by increasing lateral loading up to complete failure of the frame. Figure 12a shows a view of the frame during loading. Global force-deformation response was very similar to the frame C1 up to 10-15 kN lateral loading. For larger lateral loading, stiffness of the C2 frame was slightly larger than the one of the C1 frame. However, global resistance under horizontal loading was smaller in the case of the C2 frame. It was attained at the first local buckling in the beam near the right eaves (connection 5, see Figure 13b), when the lateral force resistance dropped suddenly. It was followed by a combined local buckling and lateral-torsional buckling of one of the columns at the mid-height (see Figure 12a and Figure 13b). Finally, local buckling of the beam at the right eaves was observed (at connection 2, see Figure 13b).

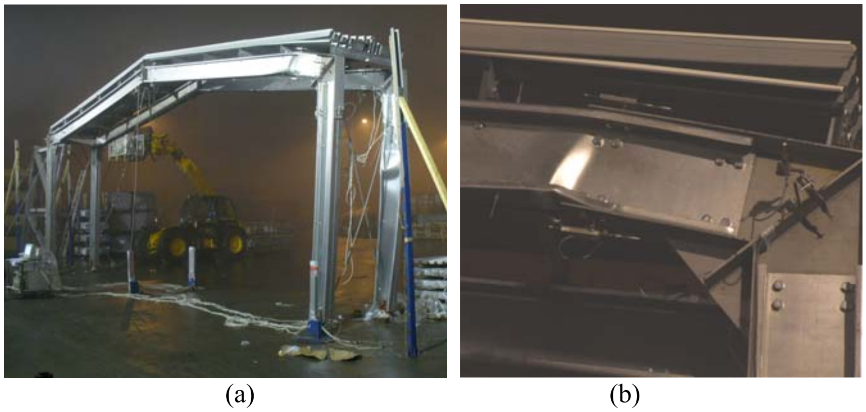


Figure 12. C2 frame: global view (a) and local buckling of the right beam connection (b).

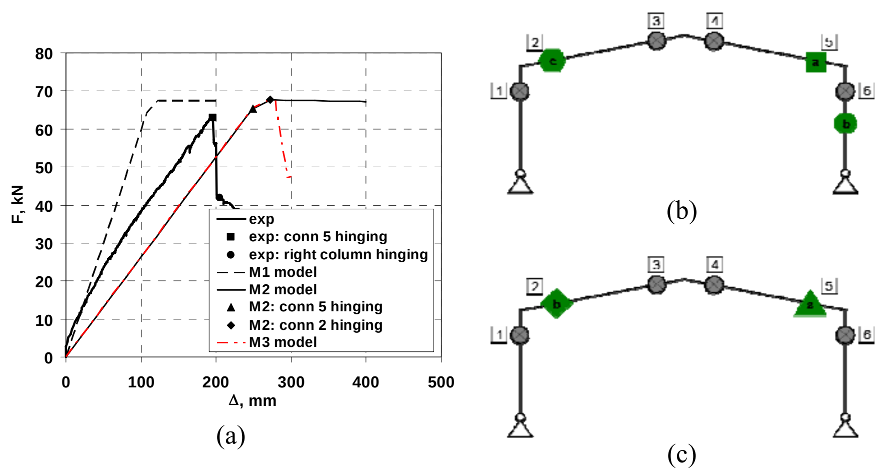


Figure 13. Frame C2: comparison of experimental and numerical lateral force - deformation curves (a), and position of local buckling observed experimentally (b) and in the numerical model (c).

A comparison of the experimental and numerical lateral force – deformation response of the C2 frame is shown in Figure 13a. As in the case of frame C1, The M1 model (with rigid connections) provides a good approximation of the initial response of the frame, up to lateral forces of about 10 kN. For larger forces, the other two models (M2 and M3), accounting for semirigid connection response, show a better approximation of experimental response for larger loads. All numerical models overestimate global frame resistance under lateral loading. This can be explained by the fact that influence of axial force was neglected when determining connection moment-rotation characteristics. Higher axial forces are present in the right column under combined effect of gravity loading and lateral loading due to the effect of overturning. While the location of first local buckling was correctly predicted by numerical model (at connection 5, see Figure 13b and c), column hinging observed in the experimental test was not confirmed by numerical models. Column hinging can be explained by neglected influence of axial force, combined with the effect of no lateral restraining at column flanges by side rails. Both of these effects were present in the experimental setup, but not in the numerical model.

It can be concluded that the M3 model seems to provide the best agreement to the experimental results, if initial stiffness, lateral resistance, and post-buckling response are envisaged. However, global frame resistance under lateral loads drops quickly after the first local buckling under maximum force is reached. Therefore, for practical cases, response to the first local buckling in members is important, which can be estimated using a more simple frame model, incorporating only the semirigid connection response, eventually an elastic perfectly plastic model. Global frame stiffness determined using bilinear moment-rotation characteristics obtained analytically by the component method is smaller than the experimental stiffness. Real initial stiffness of the connection may be higher at low moments, due to restraining provided by flanges of the bracket element and/or by the inner bolt group. A connection model capable of representing this higher stiffness would provide a closer match between experimental and numerical frame stiffness.

Conclusions

Application of the component method implemented in EN1993-1-8 for determination of connection characteristics in the case of cold-formed members is possible with a minimum number of adjustments. For the particular case of connection studied in this paper (with both flange and web bolts), its characteristics can be determined with a reasonable accuracy if only the outer bolt group of bolts is considered. The components contributing to the stiffness and strength of the connection are: cold-formed member flange and web in compression, bolts in shear, bolts in bearing on the cold-formed member, and bolts in bearing on the bracket. It is considered appropriate to use a linear distribution of forces on bolts in the case of a connection to light-gauge members.

The connection with both flange and web bolts is semirigid but full-resistant. Therefore design of light-gauge portal frames with considered type of connection need to account for connection flexibility. Connection characteristics obtained using the component method (EN1993-1-8) can be easily incorporated in the structural model, in order to obtain realistic response under lateral forces. Though a detailed moment-rotation response representing the initial stiffness, moment resistance and post-buckling response provides the most realistic global response, a simple elastic structural analysis modelling connection stiffness alone can be sufficient for design purpose. Cold-formed steel pitched-roof portal frames of back-to-back plain channel sections and bolted joints are characterised by a rapid degradation of strength after the first local buckling in its members. Therefore, frame resistance may be estimated at the attainment of the moment capacity in the most stressed cross-section using an elastic structural analysis. Axial force can reduce moment resistance of cold-formed members and need be taken into account.

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